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Waterways Division

PROCEEDINGS OF THE

AMERICAN SOCIETY

OF CIVIL ENGINEERS

VOLUME 82

THIS JOURNAL

. . . represents an effort by the Society to deliver technical data direct from the authors to the reader with the greatest possible speed. To this end, it has none of the usual editing required in more formal publications.

Readers are invited to submit discussion applying to current papers. For papers in this Journal the final date on which a discussion should reach the Manager of Technical Publications appears as a footnote with each paper.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying "Publication Procedure for Technical Paper" (Proceedings Paper 290). For free copies of this paper—describing style, content, and format—address the Manager, Technical Publications, ASCE.

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JOURNAL

WATERWAYS DIVISION

Proceedings of the American Society of Civil Engineers

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WATERWAYS DIVISION
Proceedings of the American Society of Civil Engineers

OLD RIVER DIVERSION CONTROL: THE GENERAL PROBLEM[a]

John R. Hardin,[1] M. ASCE
(Proc. Paper 906)

FOREWORD

At the Convention of the Society in St. Louis, Mo., in June, 1955, a luncheon address and four papers presented at a Joint Session of the Waterways, Hydraulics, and Soil Mechanics and Foundations Divisions formed a symposium entitled "Old River Diversion Control." Following that meeting, the several papers were reviewed by the appropriate Technical Divisions, revised to meet publication requirements, and are presented herewith under the sponsorship of the Waterways Division.

Contributors of discussion are requested to address themselves to a selected paper in the group. If more than one author is being addressed, separate discussions should be presented.

The papers are grouped under the broad title, "Old River Diversion Control." The first of these papers, "The General Problem,"(Proceedings Paper 906) by John R. Hardin, M. ASCE, presents the history of major Mississippi River diversions and analyses observations and studies leading to the conclusion that diversion of the Mississippi River through the Atchafalaya River is imminent; it is concluded that, because of the consequences, this diversion should be prevented. N. A. Graves, A.M. ASCE, in his paper "Hydraulic Requirements" (Proceedings Paper 907), cites the fact that control of the Old River has become necessary because of the threat that the Mississippi River will adopt the course of the Atchafalya River; the choice of the control site was influenced by many conditions among which are the prevailing and prospective tailwater conditions. To design the structures involved in this project, hydraulic models were used extensively according to "Use of Models" (to be published subsequently) by T. E. Murphy, M. ASCE. In "Foundation Design" (Proceedings Paper 908), W. J. Turnbull and W. G. Shockley, Members, ASCE, present the general geology and soils conditions for the low-sill, overburden, and navigation-lock structures proposed for the diversion control. "Structures Required" (Proceedings Paper 909) by Norman R. Moore, M. ASCE, is the last of the papers; for the positive control of the flow diversion, structures for flood control and a navigation lock must be provided.

Note: Discussion open until August 1, 1956. Paper 906 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, No. WW 1, March, 1956.

a. Before the American Society of Civil Engineers St. Louis, Mo., 14 June, 1955.

1. Brig. Gen., President, Mississippi River Commission, Vicksburg, Miss.

SYNOPSIS

The Atchafalaya River, joined to the Mississippi River by means of Old River, is the farthest upstream distributary of the Mississippi. It provides a route to the sea only half as long as the Mississippi's present course past Baton Rouge and New Orleans. Each year, the Atchafalaya has been diverting increasingly larger percentages of Mississippi River flow, at present about twenty-three percent. Studies conducted by the Mississippi River Commission, Corps of Engineers, in recent years, and observations made on these streams during the past seventy years, lead to the conclusion that the diversion of flow through Old River and the Atchafalaya River is approaching and will reach the "critical" stage (about forty percent of the major river's flow), at which time the parent stream is in an uncertain condition as to its future course. Additional exhaustive studies of past major diversions show that the present development is similar to those leading to earlier diversions and reinforce the conclusion that diversion through the Atchafalaya River will occur by about 1975 unless preventive measures are taken.

Should this diversion occur, the present flood control plan would have to be discarded in the lower reaches of the Mississippi and in the Atchafalaya Basin, navigation would be seriously disrupted, utilities would have to be relocated, fresh-water supplies for New Orleans would be seriously affected, and the impact upon the economy of the region would be disastrous.

To prevent diversion, the Mississippi River Commission has recommended the construction of two control structures on the west bank of the Mississippi River about ten miles upstream from Old River, a navigation lock connecting the Mississippi and Old Rivers, an earth-fill dam in Old River, approach channels for the lock, inlet and outlet channels for the control structures, and enlargement and extension of the main line Mississippi River levee in that location.

The Mississippi River has played a commanding role throughout the history of the United States. The object of attention of many of the earlier explorers, it was the primary north-south commercial artery of the infant country, the center of battles, treaties, and diplomatic maneuvers.

This great river does not often make a major change in its course to the Gulf of Mexico. It has happened before. We believe a change of wide-reaching effect would take place in the lower 300 miles within the next 20 years unless steps are taken to prevent it from doing so.

The Mississippi is truly America's principal river, serving a large and important segment of the country and looming ever larger in importance as a source of water and as a means of transportation. Great cities on its banks are growing rapidly. The river serves these cities, and through them a vast area behind them. Bridges, pipe lines, and cables span the river.

These are the physical demonstrations of the importance of the Mississippi River to the valley through which it courses and to this still-growing country. Since this river is of such importance to the nation's economy, it is obviously important that it be maintained in its present location and condition so that it may continue to play this role.

This paper is concerned with the problem of controlling the flow from the Mississippi River into the Atchafalaya, and for brevity it has been termed "Old River Control." It discusses the various factors bearing on the problem; it presents in general terms the solution proposed and now authorized

for construction. Other papers to be presented in the symposium this afternoon will discuss the details of certain phases of the problem and their solution.

The Mississippi River Basin, draining 41 per cent of the United States, is roughly funnel-shaped, converging to form a "spout" where Old River provides a connection between the Mississippi and Atchafalaya Rivers. It is through this narrow "spout" that floodwaters from an area of 1-1/4 million square miles must be carried in time of flood.

The Lower Alluvial Valley of the Mississippi River, beginning at Cape Girardeau, Missouri, and extending to the Gulf of Mexico, is the area which was subject to overflow before protective works were erected. It is through this wide, flat valley that the Mississippi River meanders in its long journey of about 1,000 river miles. The materials through which the Mississippi flows were laid down by the river itself; consequently, they are easily eroded. This condition , together with wide variations in the river's discharge and stage, creates a meandering stream whose past history is one of constant change. In building the vast delta into the Gulf, it has made several major changes in course by accepting a shorter and more attractive route to Gulf level.

The Atchafalaya River is the farthest upstream distributary of the Mississippi River. Almost unknown outside Louisiana a few years ago, today it is the third largest American river flowing into the sea. The average annual discharge of this river is increasing, and records show that this increase is due almost entirely to the progressively greater volume of Mississippi River water being diverted to it through the Old River connection.

The Atchafalaya River flows through a low, well-defined basin extending in a general north-south direction from the latitude of Old River through openings at Morgan City and Wax Lake and then out to the Gulf of Mexico, a distance of about 140 miles. On the other hand, from the same Old River the distance to Gulf level by way of the Mississippi River past Baton Rouge and New Orleans is more than twice that far, or 320 miles.

The possibility of eventual diversion of the Mississippi into the Atchafalaya has been known to engineers of the Mississippi River Commission for many years. As early as 1888, Capt. Dan C. Kingman in a report to the Mississippi River Commission stated:

> "I have given the Atchafalaya problem much thought and study. * * * The dangers to be averted and the obstacles to be overcome are, so far as the Mississippi River is concerned, * * * the deflection of its waters in ever-increasing quantities down the Atchafalaya which might in time, due to its shorter length, exceed in capacity the main stream and finally become the sole outlet of the Mississippi."

Since that time, and especially since 1928 when the present flood control project was adopted, the Mississippi River Commission has been continually studying this possibility. However, some channel enlargement has been desired, and even encouraged, so that the Atchafalaya would be in a condition to carry its share of the basin superflood upon which the project designs are based.

By 1950, the Atchafalaya River had reached a stage of development which indicated that a careful review of the situation was warranted. Enlargement of the upper river was occurring at a rate far in excess of the capacity of the

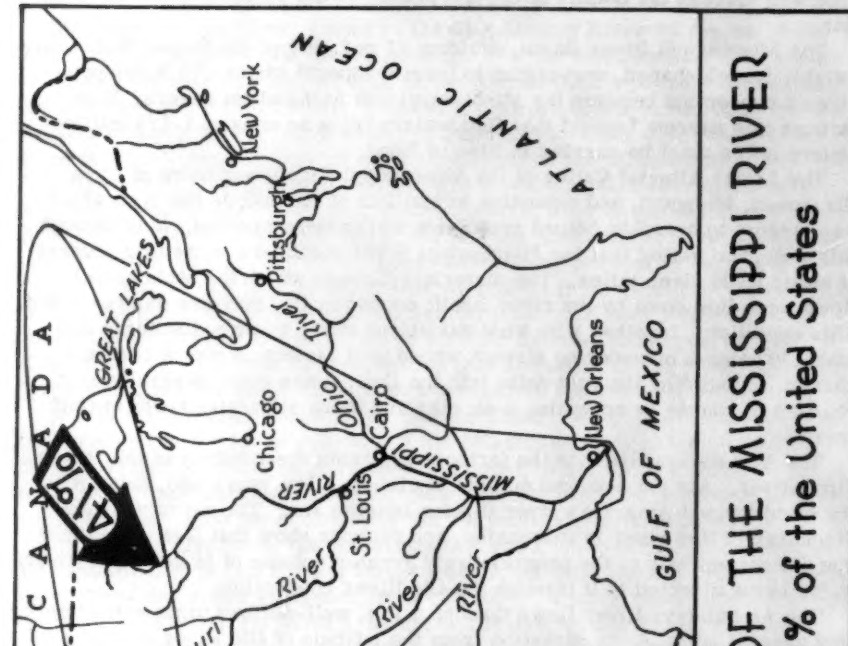
THE DRAINAGE BASIN OF THE MISSISSIPPI RIVER
The basin drains 41% of the United States
41%
CANADA
GREAT LAKES
ATLANTIC OCEAN
New York
Pittsburgh
Ohio River
Chicago
Cairo
St. Louis
MISSISSIPPI RIVER
New Orleans
GULF OF MEXICO
Missouri River
Arkansas River
Red River
Denver
Helena
Seattle
San Francisco
MEXICO
PACIFIC OCEAN

Figure 1.

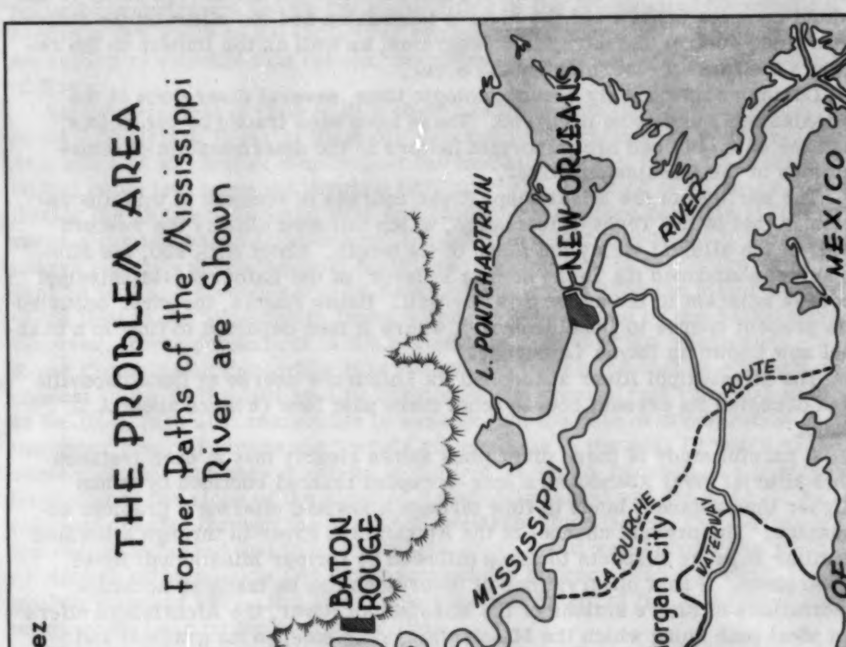
THE PROBLEM AREA
Former Paths of the Mississippi River are Shown
Natchez
BATON ROUGE
MISSISSIPPI
L. PONTCHARTRAIN
NEW ORLEANS
RIVER
LA FOURCHE
ROUTE
Morgan City
WATERWAY
INTRACOASTAL
ATCHAFALAYA RIVER
ROUTE
TECHE
OLD R.
RED
RIVER
BLACK R.
Simmesport
GULF
GULF
OF
MEXICO

Figure 2.

lower river, with a consequent danger to the flood control plan in that area. The Mississippi River Commission, therefore, began a thorough study of the whole complex problem of the river's tendencies and the effect on the authorized flood control and navigation programs, as well as the impact on the region's economy if diversion should occur.

During comparatively recent geologic time, several diversions of the Mississippi River have occurred. These have been traced by geologists, and the data obtained are important factors in the determination of future actions of the Mississippi River.

The earliest of the Mississippi River courses of concern to this discussion is that of the Teche-Mississippi, which followed closely the western wall of the alluvial valley for much of its length. About A.D. 900, the Mississippi abandoned its Teche course in favor of the Lafourche-Mississippi course adjacent to the eastern valley wall. Below Angola, the river occupied its present course to Donaldsonville, where it then departed to flow in a channel now known as Bayou Lafourche.

The Mississippi River abandoned its Lafourche course at Donaldsonville and occupied its present course from there past New Orleans about A.D. 1200.

A careful study of these diversions shows clearly that in each instance this alluvial river abandoned a long-occupied channel confined by banks higher than adjacent lands to flow through a lowland offering a gradient advantage. The present channel of the Atchafalaya River is through a lowland similar in many respects to those followed by former Mississippi River diversions. If past observations of diversions can be taken as accurate indications of future actions of the Mississippi River, the Atchafalaya offers an ideal path along which the Mississippi could steepen its gradient and shorten its course to the sea.

In each of the former Mississippi diversions, it appears that the new course taken by the river was through the route of a smaller stream flowing in a lowland near the Mississippi. Studies show that in some cases the smaller stream became a distributary of the Mississippi when a migrating meander loop intersected it, as did the Atchafalaya. Following this, the distributary enlarged until it became the new Mississippi course. In doing so it passed through distinct developmental stages.

All Mississippi River diversions appear to have occurred gradually in a span of not more than 100 years. The initial and intermediate stages are characterized by slow and progressive growth, after which a critical stage is reached when there is an essential balance between flow down the main channel and that down the diversion channel. Studies made under the Mississippi River Commission indicate that when a distributary of the main river carries approximately 40 per cent of the major river's flow the critical stage has been reached. After this, diversionary activity is greatly accelerated as the new channel develops rapidly in an attempt to reach a state of equilibrium.

In studying the various aspects of the problem to determine the probable time at which diversion might occur, consideration was given to the rate at which the Atchafalaya River has been pushing through the deposits being dropped in the lower portion of the Atchafalaya Basin known as Grand Lake and Six-Mile Lake, just north of Morgan City, La. It was estimated that if the downstream advancement of the river channel which occurred from 1916 to 1950 were continued, an efficient single river channel would be developed

by 1985 or 1990. The existence of such a well-developed channel is an important consideration before the major flows of the Mississippi River will choose a new route to the Gulf. For reasons presented later, it will be necessary to expedite this channel development by dredging and confinement of flows.

Another element of time in this study has to do with the discharge capacity of the Atchafalaya River in its upper reaches in the vicinity of Simmesport. At a stage of 40 feet at Simmesport the discharge of the river increased 60,000 cubic feet a second between 1892 and 1932, a period of 40 years. During the 18 years between 1932 and 1950, the discharge capacity at the same stage increased an additional 130,000 cubic feet a second. If the discharge capacity at Simmesport continues to accelerate in the future as in the past 18 years, a condition will be developed where by 1968 the major portion of the flow of the Mississippi River will have been seized by the Atchafalaya. However, it was pointed out in the engineering report of the Mississippi River Commission that, while it is reasonable to assume that an efficient channel to the Gulf might well be developed at a greater rate than that shown in the past, it is also reasonable to assume that the rate of acceleration of the upper river at Simmesport would not continue in the next 18 years at the same rate that had been observed in the last 18 years, because of a general flattening of the slope of the river in its leveed channel and the need for an efficient channel through the basin for moderate flood flows. In conclusion, the report indicated that the Atchafalaya River could well become the main or master stream below Old River somewhere between 1968 and 1985, probably around 1975. But this was not the full extent of the features of the study which bear on the element of time.

Channel dimensions of the Atchafalaya River from its head to the end of the leveed portion of the river were studied in great detail and the rate of enlargement was noted. The rate of enlargement of Old River was given attention, as were the changes in cross-sectional area of the Mississippi River above and below Old River. The changes in stage-discharge relationship were studied intensively for all of the various river channels under consideration. The changes in slopes of the river surfaces and the deepening of the upper portions of the Atchafalaya into the easily erodible sands had some bearing on the conclusions reached. The geology of the river basin and the previous and indicated future changes of the river's pattern gave a general insight into the sequences in the phenomena of the river's flow.

The diversion of a river's flow into a new channel does not occur suddenly. The forces of nature have to work for considerable periods of time to scour a channel which is large enough and efficient enough to entice a river to adopt a new course. Huge quantities of material have to be transported out to the sea or deposited in overbank areas.

When the critical stage is reached, the rate of enlargement may be very rapid, and within a few years the channel could become so large it would be beyond control. At the present time the Atchafalaya is carrying on an average of 23 per cent of the Mississippi's flow at Old River, well below the flow required for the critical stage of diversion. However, this percentage is increasing and its rate of increase could become accelerated by a series of large floods such as were experienced in 1927, or even 1950, and result in the critical stage occurring about 1965.

Using as a background these exhaustive engineering and geologic studies, the many years of records and personal observation, and the opinions of some

of the nation's outstanding engineering consultants, the Mississippi River Commission reported to the Chief of Engineers, U.S. Army, that:

> "The Atchafalaya River, if left alone, will capture the Mississippi. * * * How soon this will occur is not susceptible of precise determination. It would be unwise to remain unprepared to take prompt and effective steps to prevent such an occurrence."

It appears that all of the elements present in the concept of diversions are now present in the Old-Atchafalaya situation and in a pattern favorable to the Mississippi's accepting the Atchafalaya channel as its own within a relatively brief span of years. If, as expected under normal conditions of the development, the Atchafalaya becomes the master or larger stream in about 1975, the change will have gone practically beyond control. The problem of controlling flow and of taking corrective action should be approached on the basis of occurrence of the critical stage rather than when diversion will likely be an accomplished fact.

Any plan for control of flows in the vicinity of Old River must be based upon a number of factors. One of the most important is retention of the present, proven, flood control plan, which was authorized in 1928 following the disastrous flood of 1927. This plan is a blend of a number of features, each to perform a specific function, all of which, when completed, will protect the alluvial valley of the lower Mississippi River from the greatest flood which meteorologists believe can be expected to occur. The flood is greater than that of 1927: at the latitude of Old River, it is approximately 3,000,000 cfs; below the Morganza Control Structure, the safe capacity of the leveed Mississippi River is 1,500,000 cfs, and 250,000 cfs of this is drawn off, under the plan, before it reaches New Orleans. This leaves 1,500,000 cfs which must be disposed of at the latitude of Old River.

During time of project flood, 900,000 cfs are to enter the head of the Atchafalaya Basin, 650,000 of which are to flow down the Atchafalaya River and 250,000 down the West Atchafalaya Floodway. The remaining 600,000 of the quantity to be diverted is to be carried down the Mississippi River about 20 river miles and passed through the Morganza Floodway. All the diverted flow—a total of 1,500,000 cfs—will meet at the lower end of the Atchafalaya River guide levees and will pass through the lower Atchafalaya Basin and be discharged into the Gulf of Mexico through two outlets.

The development of the comprehensive flood control plan is well advanced. On the Mississippi River below Old River, the authorized flood control works are essentially complete, and the Mississippi River channel, with its present confining levees, can accommodate the portion of the project flood for which it is designed.

In the Atchafalaya Basin, many of the features have been constructed and the entire project is about 85 per cent complete. However, maintaining the Atchafalaya Basin Protection Levees to the designed grade is extremely difficult, owing to poor foundation conditions and levee subsidence.

For proper functioning of the flood control plan, efficient channels must be developed and maintained in both the Mississippi and the Atchafalaya Rivers. Any appreciable deterioration in efficiency of either stream would have very serious consequences. Under present conditions, the lower Atchafalaya Basin is filling with sediment at a rapid rate. Siltation from Red and Mississippi River waters contributes to this action, but the scouring of the bed and banks of Old River and upper Atchafalaya River is probably the

principal source of deposition. The filling of the lower Atchafalaya Basin is, of course, causing the flood flow line to rise. Since the levees in this area cannot be raised with assurance, due to the inadequate foundation conditions in various reaches, the lower Atchafalaya River must be increased in capacity by dredging and confinement. Yet to do so without controlling the diversion tendency of the Mississippi would only be inviting disaster. Therefore, the initiation of control works at the head is necessary before relief of a critical situation in the lower Atchafalaya Basin can be undertaken.

The Mississippi River is becoming increasingly important as a navigation artery. It is the main stem of a vast network of inland waterways, spreading through the central United States and connecting the Great Lakes to the Gulf. Crossing the Mississippi at New Orleans is the Gulf Intracoastal Waterway along which many strategically important commodities are moved in great quantity. It is the route of the ever-increasing barge movement of refined petroleum products from the Texas and Louisiana refineries to the Mississippi, Missouri, and Ohio basins, and is also the principal means of access to the rapidly growing off-shore petroleum exploration and development operations.

In recent years, the Atchafalaya River, together with Old River, has become an important navigation artery. For the new high-powered, streamlined tows seeking the shortest route from Texas to points in the Upper Mississippi Basin, this route has many advantages, is 172 miles shorter than by way of New Orleans, and avoids the congestion in New Orleans harbor. It is reasonable to assume that traffic will continue to increase on the Atchafalaya River.

It is obvious that any major change in the lower Mississippi River which would alter the efficiency of the flood control plan or the ability of the Mississippi and its network of navigable streams to support the continually increasing requirements of commerce would have great impact on the economy of the region.

Uncontrolled diversion of the Mississippi River through the Atchafalaya would result in a complete change in river channels, with the present Mississippi channel below Old River eventually being closed except for flood flows. Existing navigation patterns would undergo great changes; new navigation structures would be required and old ones modified. The usefulness of existing flood control structures, notably the Bonnet Carre and Morganza spillway structures, would be impaired or destroyed. Communication networks would have to be rebuilt. Salt-water intrusion would exist as far upstream as Baton Rouge, with tremendous, if not disastrous, effects on the industrial and domestic use of water.

The results of such a diversion would not be restricted to the area below Old River. Equilibrium conditions now existing in the Mississippi River would be disturbed as far upstream as Vicksburg, and an increase in meander activity would no doubt result, making obsolete a large part of the now-effective bank stabilization works and threatening the almost complete main line levee system. In addition, navigation during the period of adjustment would be extremely difficult because of increased velocities and stream meandering.

It is evident that should the Atchafalaya be permitted to capture the Mississippi the effects would be far-reaching and of vast consequence, seriously and permanently altering the way of life of a large and prosperous region.

The plan for preventing the diversion does not include changing any existing condition but instead is designed to retain what now exists. Any works constructed for the prevention of uncontrolled diversion must provide for the regulation of flows to satisfy the many exacting requirements in preservation of present conditions. It appears to be necessary, then, to provide permanent-type control structures so that flows may be regulated as required and so that maintenance of the desired water-sediment relationship may be assured.

Any plan for the control of flows at Old River will require the location of structures in Old River, in the Atchafalaya River, on the west bank of the Mississippi River above Old River or at a combination of these locations. Regardless of location, the structure or structures must be capable of passing the project flood flows in the amounts already specified in the flood control plan; they must prevent permanent capture of the Mississippi by the Atchafalaya; they must be flexible in operation in order to maintain the present distribution of flows of water and sediment as nearly as possible. The plan arrived at must provide for navigation, it must offer assurances of long life and it must be reasonable in first cost and subsequent maintenance. The nature of the problem and its importance to the national economy require a safe and permanent solution.

A number of plans were considered. Among these were plans calling for the location of major structures in Old River; plans requiring structures located in the Atchafalaya River near the lower end of the guide levees; plans requiring dams across the Atchafalaya River in its upper reaches, in conjunction with weirs, overflow dams, and notches; plans requiring construction in the Atchafalaya of a series of low-sill dams of stone or other nonerodible materials; and plans to achieve friction control by increasing channel roughness. All of these, investigated in as much detail as necessary, were discarded because they failed to meet the criteria set forth.

The remaining possibility was to locate the proposed structures where flow from the Mississippi River could be controlled before it reached the Red River backwater area. As this area now acts as a distributing reservoir for overbank flows at this latitude, structures located to take advantage of this reservoir without altering present conditions would be ideal from both an engineering and an economic viewpoint. Control structures on the west bank of the Mississippi River above Old River, with a lock in Old River, would satisfy the conditions of effectiveness, permanence, safety, and reasonable cost. This, generally, is the plan proposed for construction.

The plan calls for two control structures on the west bank of the Mississippi River about ten miles upstream from Old River, a navigation lock connecting the Mississippi and Old Rivers, an earth-fill dam in Old River, approach channels for the lock, inlet and outlet channels for the control structures, and enlargement and extension of the main line Mississippi River levee in that location. The two control structures are planned to accommodate a flow of approximately 700,000 cfs from the Mississippi River during the project flood.

The smaller of the two control structures will be a low-sill structure of reinforced concrete, 566 feet long between abutments. It will have eleven gate bays, each having a 44-foot clear opening between piers. The weir crests will be at such elevations as to permit flows at all stages to pass from the Mississippi River into an outflow channel extending to Red River.

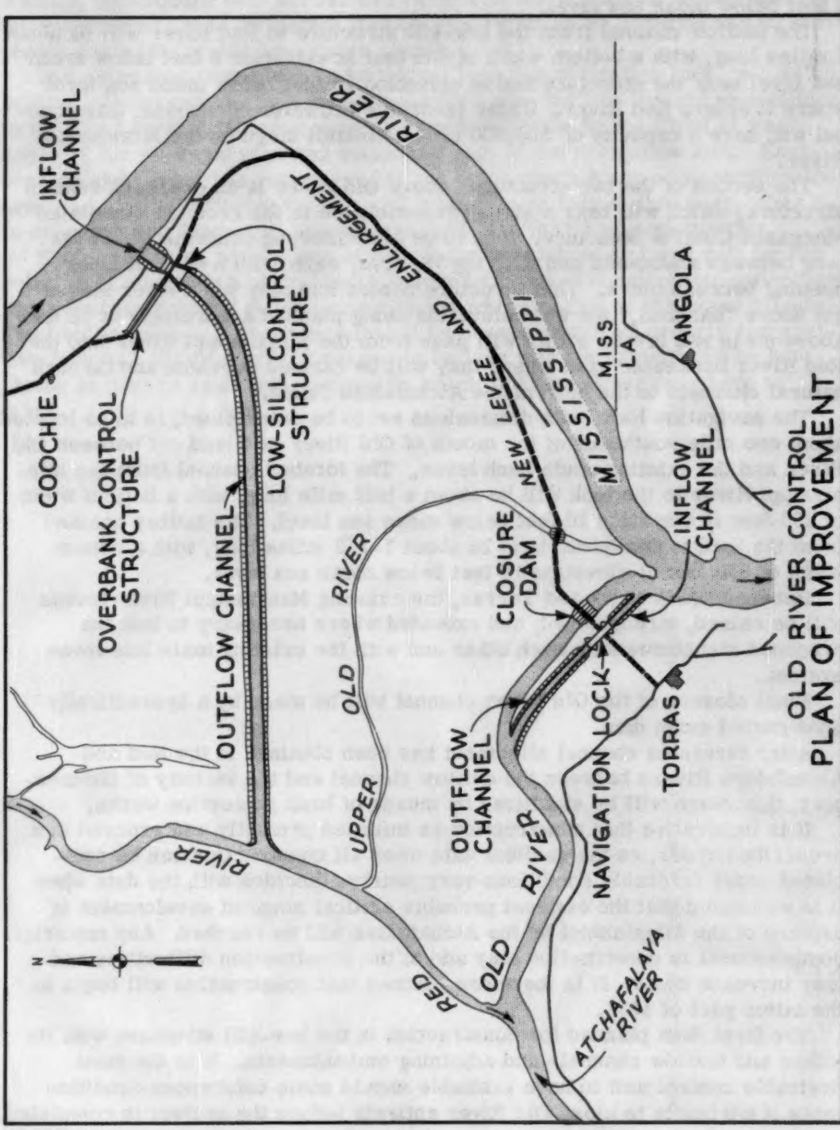
INFLOW CHANNEL
COOCHIE
OVERBANK CONTROL STRUCTURE
LOW-SILL CONTROL STRUCTURE
OUTFLOW CHANNEL
RIVER
NEW LEVEE AND ENLARGEMENT
MISSISSIPPI
MISS
LA
ANGOLA
INFLOW CHANNEL
CLOSURE DAM
UPPER OLD RIVER
OUTFLOW CHANNEL
NAVIGATION LOCK
TORRAS
OLD RIVER CONTROL
PLAN OF IMPROVEMENT
RED RIVER
OLD RIVER
ATCHAFALAYA RIVER
N

Figure 3.

The inflow channel from the Mississippi River to the low-sill structure will be about a half mile long, with a bottom width of 1,000 feet at elevation 5 feet below mean sea level.

The outflow channel from the low-sill structure to Red River will be about 7 miles long, with a bottom width of 900 feet at elevation 8 feet below mean sea level near the structure and at elevation 10 feet below mean sea level where it enters Red River. Under favorable tailwater conditions, this channel will have a capacity of 300,000 cfs at bankfull stage on the Mississippi River.

The second of the two structures above Old River is an overbank control structure, which will bear a strong resemblance to the recently completed Morganza Control Structure. It is to be of reinforced concrete, 3,356 feet long between abutments and having 73 bays, each with a 44-foot clear opening between piers. This structure comes into play when river stages get above "bankfull," the weir elevation being planned accordingly at 52 feet above mean sea level. Flows will pass from the Mississippi River into the Red River backwater area where they will be carried overland and through natural channels to the head of the Atchafalaya Basin.

The navigation lock, with dimensions yet to be determined, is to be located about one mile southwest of the mouth of Old River in a land cut between Old River and the existing south bank levee. The forebay channel from the Mississippi River to the lock will be about a half mile long, with a bottom width of 250 feet at elevation 10 feet below mean sea level. The tailbay channel from the lock to Old River is to be about 1-1/2 miles long, with a bottom width of 250 feet at elevation 10 feet below mean sea level.

Between Black Hawk and Torras, the existing Mississippi River levees will be raised, strengthened, and extended where necessary to link the proposed structures with each other and with the existing main line levee system.

Final closure of the Old River channel will be made by a hydraulically constructed earth dam.

After favorable channel alignment has been obtained in the Red and Atchafalaya Rivers between the outflow channel and the vicinity of Simmesport, this reach will be stabilized by means of bank protection works.

It is imperative that construction be initiated promptly and proceed in a prescribed order, as the earliest date when all construction can be completed under favorable conditions very nearly coincides with the date when it is estimated that the earliest probable critical stage of development in capture of the Mississippi by the Atchafalaya will be reached. Any material postponement in construction may add to the construction difficulties and may increase costs. It is therefore planned that construction will begin in the latter part of 1955.

The first item planned for construction is the low-sill structure with its inflow and outflow channels and adjoining embankments. It is the most desirable control unit to have available should some unforeseen condition make it advisable to close Old River entirely before the project is completed.

The overbank control structure and upper levee embankments should be completed next. Then the overbank gap at Old River can be reduced in width or closed entirely if necessary to retard or prevent a possible dangerous acceleration in channel enlargement of the Old and Atchafalaya Rivers.

Next, it is planned to build the navigation lock and its approach channels. Finally, the closure dam across Old River and the remaining levees should be constructed. It is important that these structures be undertaken and completed during favorable low-water stages.

The estimated cost of the undertaking, less the navigation lock, is $47 million. Congress has authorized the entire plan but has limited expenditures to the flood control features pending a determination by the Chief of Engineers of the required size and estimated cost of the navigation lock. Engineering and design has progressed to the point where plans and specifications are ready for advertising the low-sill structure in July of this year, when it is expected that funds will become available for initiating construction.

The undertaking will be one of very large magnitude and will call for the solution of many unusual problems in foundations and construction procedures. The great importance of the project to the whole region of the lower Mississippi valley makes it necessary that it be prosecuted in an orderly and positive fashion. There is every reason to believe that the project can be completed in time to meet any maneuvers which Old Man River may have in mind.

JOURNAL

WATERWAYS DIVISION

Proceedings of the American Society of Civil Engineers

OLD RIVER DIVERSION CONTROL: HYDRAULIC REQUIREMENTS

E. A. Graves,[1] A. M. ASCE
(Proc. Paper 907)

FOREWORD

At the Convention of the Society in St. Louis, Mo., in June 1955, a luncheon address and four papers presented at a Joint Session of the Waterways, Hydraulics, and Soil Mechanics and Foundations Divisions formed a symposium entitled "Old River Diversion Control." Following that meeting, the several papers were reviewed by the appropriate Technical Divisions, revised to meet publication requirements, and are presented herewith under the sponsorship of the Waterways Division.

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The papers are grouped under the broad title, "Old River Diversion Control." The first of these papers, "The General Problem," (Proceedings Paper 906) by John R. Hardin, M. ASCE, presents the history of major Mississippi River diversions and analyses observations and studies leading to the conclusion that diversion of the Mississippi River through the Atchafalaya River is imminent; it is concluded that, because of the consequences, this diversion should be prevented. N. A. Graves, A. M. ASCE, in his paper "Hydraulic Requirements" (Proceedings Paper 907), cites the fact that control of the Old River has become necessary because of the threat that the Mississippi River will adopt the course of the Atchafalaya River; the choice of the control sile was influenced by many conditions among which are the prevailing and prospective tailwater conditions. To design the structures involved in this project, hydraulic models were used extensively according to "Use of Models" (to be published subsequently) by T. E. Murphy, M.ASCE. In "Foundation Design" (Proceedings Paper 908), W. J. Turnbull and W. G. Shockley, Members, ASCE, present the general geology and soils conditions

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Note: Discussion open until August 1, 1956. Paper 907 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, WW 1, March, 1956.

1. Hydr. Engr., Mississippi River Commission, Corps of Engrs., U. S. Army, Vicksburg, Miss.

for the low-sill, overburden, and navigation-lock structures proposed for the diversion control. "Structures Required" (Proceedings Paper 909) by Norman R. Moore, M. ASCE, is the last of the papers; for the positive control of the flow diversion, structures for flood control and a navigation lock must be provided.

SYNOPSIS

The streams and floodways of the area form a complex system. Observed and planned discharges illustrate the magnitudes involved. The control of Old River has become necessary due to the threat that the Mississippi will adopt the course of the Atchafalaya. The selection of a site for the control was influenced by a number of factors. An unusual situation exists with respect to the determination of prevailing and prospective tailwater conditions. The expected discharges through the structures are compared with natural flows. The structures will permit of regulation if required to reduce the rate of enlargement of the Atchafalaya River and will give assurance against the change of course of the Mississippi River.

INTRODUCTION

The Old-Atchafalaya River system is the farthest upstream distributary of the Mississippi. The bifurcation is at a point about 313 miles above the mouth of the parent stream. The distance from the departure point to the mouth of the Atchafalaya River is only about 140 miles; thus the slope through the Old-Atchafalaya system is more than twice that found in the Mississippi below their common point. A raft of logs which blocked the Atchafalaya from its head for a distance of perhaps 20 miles downstream was removed prior to 1855. Over the past 90 years the enlargement of the Atchafalaya River, a desirable development from the standpoint of accommodating flood flows, has been viewed with concern by those who feared that it might result in the change of course of the Mississippi River. This latter event now seems increasingly probable unless preventive measures are taken. Beyond the scope of this paper, a number of alternative plans have been considered over the years and also recently while the adopted plan was being developed. The present paper describes the hydraulic considerations which control the design of the adopted plan.

Stream and Basin Description

Old River, which follows the lower arm of a cutoff meander loop of the Mississippi, extends for six miles from its junction with the Mississippi to a point where it joins the Red River to form the head of the Atchafalaya River proper. A project plan map of the area on which the plan to be described has been superimposed is shown on Fig. 1. In Old River, channel depths vary from 60 to 130 feet. A discussion of the enlargement and increased natural diversion at Old River has been given elsewhere.(1) At the junction of Old River with the Mississippi there is a gap of about three miles in the Mississippi River levee, and for a distance of about 14 miles upstream to the vicinity of Black Hawk the levee is low enough

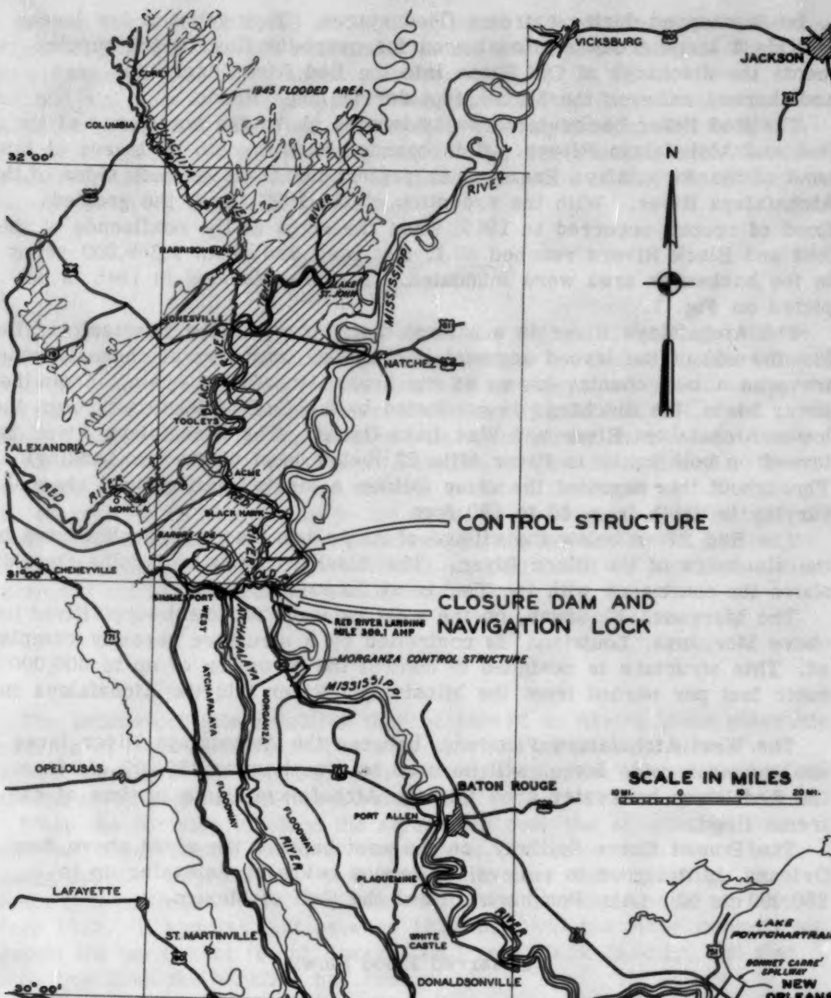

FIGURE I - PROJECT PLAN

to be overtopped during extreme flood stages. This gap and low levee provide a large cross-sectional area for overbank flow, which supplements the discharge of Old River into the Red River backwater area, and thereby relieves the Mississippi during large floods.

The Red River backwater area is located above the confluence of the Red and Atchafalaya Rivers. It is bounded on the south by levees at the head of the Atchafalaya Basin which protect the lands on both sides of the Atchafalaya River. With the exception of the 1927 flood, the greatest flood of record occurred in 1945, when the stage at the confluence of the Red and Black Rivers reached 60.1, feet msl, and about 1,300,000 acres in the backwater area were inundated. The area flooded in 1945 is depicted on Fig. 1.

The Atchafalaya River is a natural distributary of the Mississippi. Below the end of the leveed segment it branches into several channels which traverse a lake country known as the lower Atchafalaya basin. From the lower basin, the discharge is conducted to the Gulf of Mexico through the lower Atchafalaya River and Wax Lake Outlet. The Atchafalaya River is leveed on both banks to River Mile 52 (below head at Barbre Landing). Throughout this segment the river follows a single, well-defined channel varying in depth from 80 to 180 feet.

The Red River below the village of Acme has its flow supplemented by the discharge of the Black River. The Black River is called the Ouachita above its confluence with the Tensas at Jonesville (Fig. 1).

The Morganza Floodway, on the west bank of the Mississippi River just above Morganza, Louisiana, is controlled by a structure recently completed. This structure is designed to control the diversion of up to 600,000 cubic feet per second from the Mississippi River into the Atchafalaya Basin.

The West Atchafalaya Floodway, between the Atchafalaya River levee and the west guide levee, will be used to divert up to 250,000 cfs from the Red River backwater area into the Atchafalaya Basin in time of extreme flood.

The Bonnet Carre Spillway, on the east bank of the river above New Orleans, is designed to relieve the levees below by releasing up to 250,000 cfs into Lake Ponchartrain and the Gulf of Mexico.

Observed Flood Flows

Maximum recorded discharges are 1,515,000 cfs in the Mississippi River below Old River in 1945, 1,977,000 cfs in the Mississippi River above Old River in 1937, 514,000 cfs in Old River in 1937, and 661,000 cfs in the Atchafalaya River at Simmesport in 1945. An observation of 1,595,000 cfs was made in the Mississippi River below Old River in 1882; however, the amount of discharge was influenced by levee crevasses.

Flow Distribution for Project Flood

The plan for flood control in the vicinity of Old River is based on handling

a project flood of 3,000,000 cfs, half of which is to be carried in the Mississippi River and the remainder in the Atchafalaya Basin. The flow distribution is illustrated schematically on Fig. 2. A flow of about 700,000 cfs from the Mississippi River into the Red River backwater area is required to effect the flow distribution contemplated by the flood control plan. About 100,000 cfs is planned to go into storage; thus the diagram (Fig. 2) shows 600,000 cfs as the contribution of the Mississippi to the outflow from the backwater area.

The Plan for Control

The general layout of the plan is shown on Fig. 4. Discharge will be carried through controlled inlet structures to be located in the levee above Point Breeze. The control structures will be comprised of two units: A controlled overbank structure with crest at elevation 52 feet msl, and a gated low-sill structure having three bays with crest elevation at minus five feet msl and eight bays with crest elevation at plus 10 feet msl. The overbank structure will provide for the relief of floodwaters. The low-sill structure will also assist in the relief of floodwaters, but will be made deep in order to permit discharge during moderate and low stages of the Mississippi River. The low-sill structure will be provided with an excavated channel leading to Red River.

Site Selection

The process of site selection may be viewed as having taken place in a series of separate steps. The area near the Mississippi River was chosen because of the much greater ease of protection against loss of the structures by degradation downstreamward.

When the decision to place the structures near the Mississippi River bank had been reached the next step was to find a location where bank caving appears improbable. The chosen site near Knox Landing, La., meets this criterion. Fig. 6 shows bank lines of a number of surveys since 1823. It appears that between 1823 and 1883 the river moved over against the hard point (Point Breeze, La.) near Knox Landing, and that since that time the bankline has been stable.

Another consideration of importance is that the chosen location should be such that the diverted water will contain bed-sediment in about the proportion that obtains in the Mississippi River. Analytical and model studies indicate that this location should be satisfactory in this regard. It is in a relatively straight stretch of river, and the cross-section (Fig. 6) has since 1895 been wide, shallow, and relatively flat. Reasoning based on such considerations, and confirmed by measurements, indicates that the distribution of sediment is relatively uniform across the section.

Two sites farther upstream were considered and discarded. One, near Coochie, La., is subject to bank caving and appears less favorable for the diversion of bed load. Another, in the vicinity of Black Hawk, is about op-

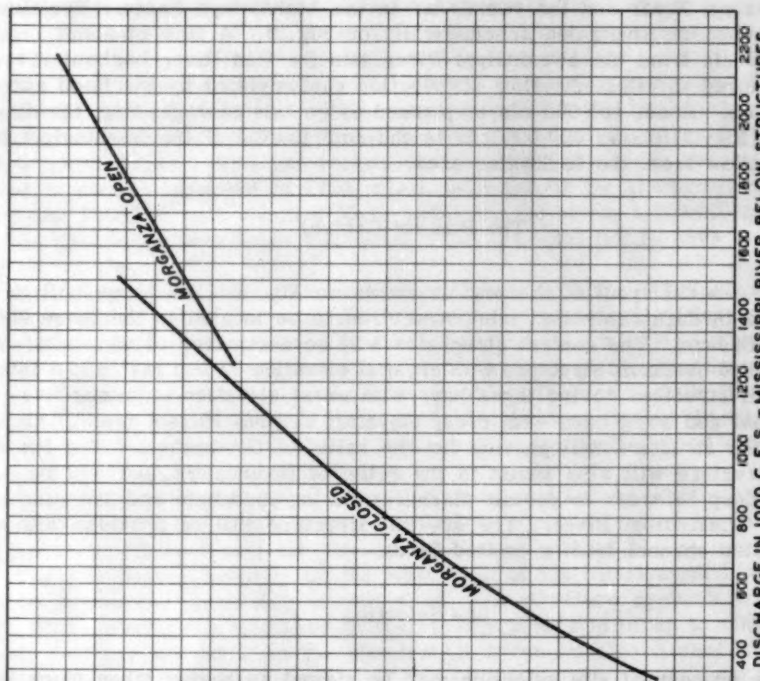

FIGURE 3 - HEADWATER CURVES FOR CONTROL STRUCTURES

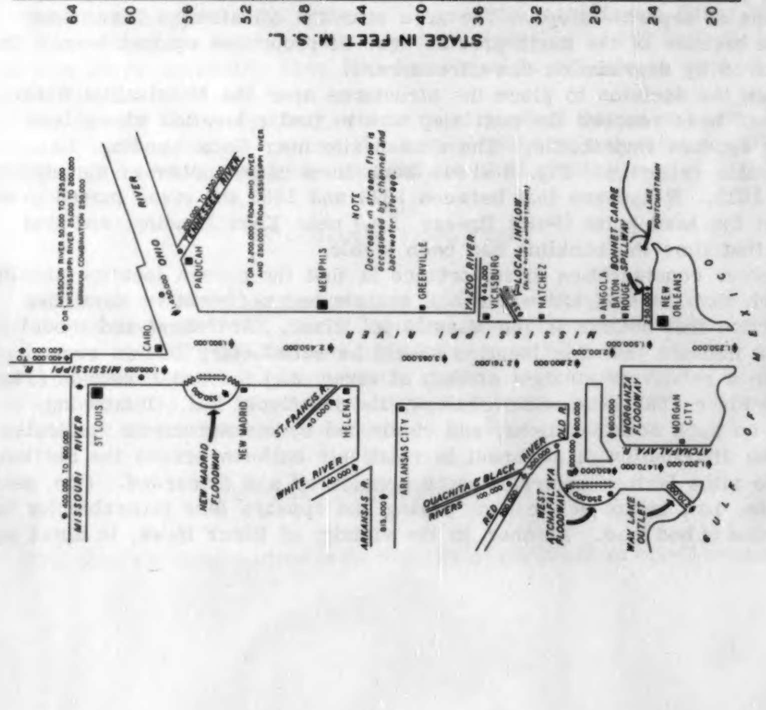

FIGURE 2 - FLOW DISTRIBUTION FOR PROJECT FLOOD

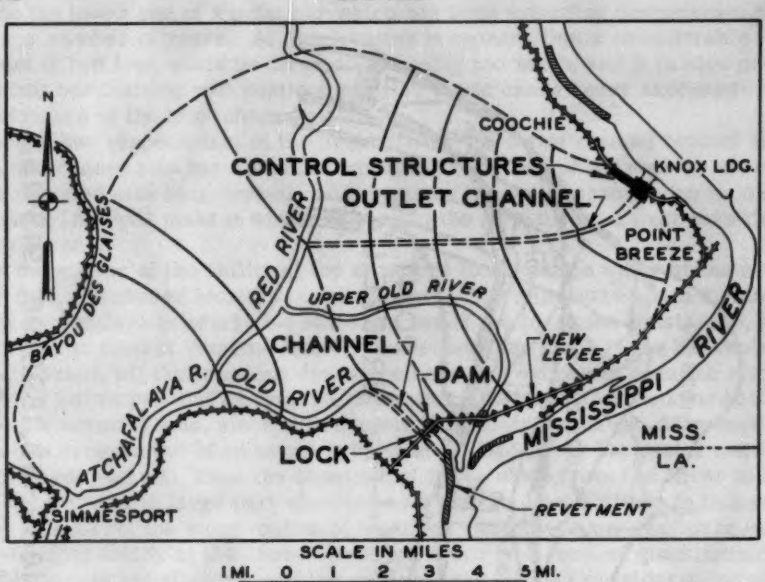

FIGURE 4 - GENERAL LAYOUT OF PLAN AND FLOW DIAGRAM

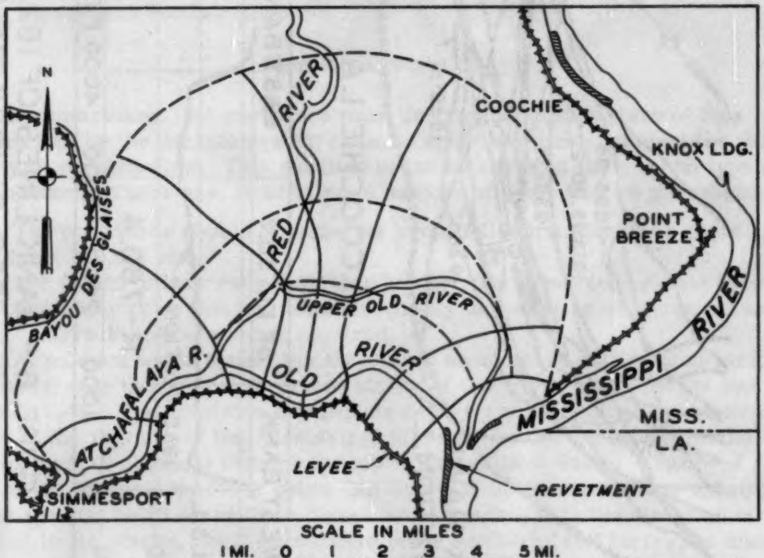

FIGURE 5 - FLOW DIAGRAM, NATURAL CONDITIONS, MODERATE FLOOD

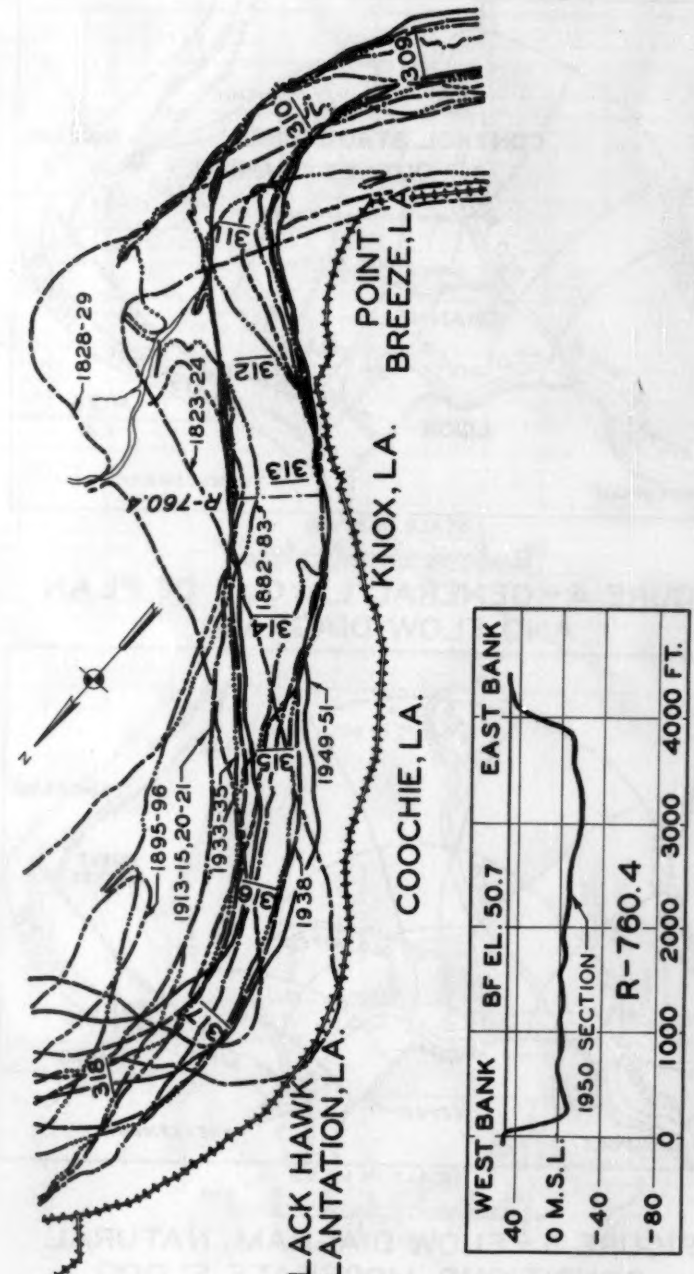

FIGURE 6 - BANK LINES OF 1823-24, 1882-83, 1895-96, 1913-15, 1920-21, 1933-35, 1938 AND 1949-51

posite the lower end of a point bar which has been extending downstream slowly for a number of years. At this location it appears that a considerable amount of bed load would be diverted, probably too much, and it is also probable that bar building will continue and this would cause heavy expense for maintenance of the inlet channel.

In another respect, that of the protection of the outlet channel against erosion, the chosen site has a marked advantage. A study of the geology of the area revealed clay plug deposits across which the outlet channel can be made to pass. This will make it easier to protect the structures against ravelling of the channel.

In the matter of the ability of the structure to discharge water at the necessary rate, the chosen location is satisfactory. Since the water-path distance to the Atchafalaya is nearly the same for any of the locations considered, and also since in time of extreme flood the water surface slope in the backwater area is small, all the locations considered are approximately equal as regards height of tailwater. But as regards headwater, the more upstreamward locations are advantageous, since with project flood distribution the difference in elevation between the Mississippi River and the Red River backwater near Old River is very small. Thus the Mississippi River slope from Old River to the structure site is in large part effective as a gain in input ability. In this respect, therefore, the more upstream locations would have been advantageous. But the input ability at the chosen location, which is discussed quantitatively hereinafter, is satisfactory, and the several very weighty considerations above enumerated outweigh the hydraulic advantage.

With the choice narrowed to a small area there remained the selection of a site having the best foundation conditions in this vicinity. These studies are being reported in another paper.(2)

Flow Requirements

The requirement that assurance must be given against capture of the Mississippi by the Atchafalaya River is, in one view, a requirement for the ability to regulate flow. This resulted in the selection of the general type of plan adopted. There are, in addition, a number of other flow requirements:

a. The structures should provide, for project flood, a discharge capacity of at least 700,000 cfs.

b. For floods of less magnitude than project, the structures should provide sufficient capacity so that increased frequency of operation of Morganza and Bonnet Carre Floodways is not required.

c. At medium water stages the structures should have sufficient capacity to approximate the natural diversion ability of Old River in the period just prior to 1950. The provision of adequate capacity will assist in the maintenance of the regimen of the Mississippi River below Old River, and in the development of channels through the lower Atchafalaya Basin. Change of regimen of the Mississippi River below Old River, if in the direction of causing higher stages, could necessitate higher levee grades; if in the direction of causing lower stages, could necessitate levee set-backs and increased maintenance of bank protection works, and could reduce the intake capacity of Morganza Floodway.

d. The structures should not reduce low water flow to such a degree as to be harmful to navigation or water use in the Atchafalaya Basin.

Headwater Curves for the Structures

In order to appraise the discharge performance of the structures it is necessary to consider the Mississippi River stages which will constitute the headwater on the structures. Excellent gage and discharge records are available for Red River Landing, an important gaging station at Mississippi River Mile 300.1 AHP, about 3/4 mile below the mouth of Old River. The rating curve for 1950 conditions has been used. Computed flowlines and the results of model tests have been used to extend the rating curves for 1950 conditions, assuming Morganza operating. Starting with stages from this rating curve and assuming Old River closed, flowlines were run to the proposed structure location. Channel friction factors were obtained from flowline computations for observed conditions. For high stages the estimated overbank "n" value was Manning's 0.150. The headwater curves thus developed are shown on Fig. 3.

Under existing conditions the discharge capacity of the Mississippi River is greater above Old River than in the reaches below. This is because the channel above has had to accomodate itself to carry the total discharge which, at Old River, is divided between that stream and the Lower Mississippi River. It is expected that after the structures have been operating for several years, the capacity of the Mississippi River channel between Old River and the site of the structures will be reduced to about that which now exists below Old River. It is therefore indicated that the headwater curves, which are based on existing channel conditions, will give slightly lower headwaters than may be expected to obtain after the structures have been in operation for an indefinite period of years and the channel below has had opportunity to adjust itself.

Mississippi River rating curves are subject to an irregular periodic fluctuation, being alternately somewhat higher and lower than average. Such a fluctuation has been observed at Red River Landing. A study of discharge records since 1893 shows that in 1893, and 1911-1912 the average rating curves were virtually the same as existed in 1950. These rating curves give stages as low as any that have occurred in other years for the same discharges, and consequently the use of 1950 conditions headwater curves is expected to result in conservative estimates of the discharge capacity of the structures.

Maximum Probable Tailwater Conditions

These are assumed to be the conditions which existed in 1950. Excellent stage and discharge records exist for Simmesport, at the upper end of the leveed Atchafalaya River. Flowlines from Simmesport to the mouth of Old River have established that in Old River the Manning's "n" value averages about 0.030 at bankfull stages and is about 0.035 at higher stages. A flowline for the crest of the 1950 flood indicates that the overbank Manning's "n" value for the backwater area is 0.175, obtained simultaneously with an Old River channel value of 0.035. The flow diagram for this condition is shown on Fig. 5.

Assuming Old River closed, flowlines were run from Simmesport to the structures through Red River, the outlet channel, and the overbank. The flow

diagram for this condition is shown on Fig. 4. The Manning's "n" values were for Red River 0.030, for the outlet channel 0.030, and for the overbank 0.175. Elements for Red River are from a survey made in 1950. The outlet channel has been assumed to have a bottom 900 feet wide at minus 8 feet msl, sloping to minus 10 at Red River; this approximates the elevation of the bottom of Red River in this vicinity.

The nature of the backwater is such that the tailwater stage depends on the flow from the tributary streams as well as on the flow from the structures. For this reason the curves take the form of a family (Fig. 7). From the measurements of many years the tributary inflows were plotted against the concurrent Mississippi River discharges, and with this plot as a guide an envelope curve embodying the maximum probable concurrent contributions of the tributaries was drawn across the family of tailwater curves. This curve is useful for defining the upper limits of tailwater for which the stilling basins need to be designed.

Minimum Probable Tailwater Conditions

In approaching the problem of estimating the future tailwater conditions, it is necessary to consider the changes that are likely to affect the tailwater. Changes that may occur in the lower Atchafalaya Basin, below the end of the river levees (see Fig. 1), were considered but it was found that they are not likely to cause significant changes in the tailwater conditions for the following reasons: Below the end of the river levees, most of the flood flow is carried overbank within the floodway. There has been a heavy deposit of sediment in this overbank area, causing higher stages for given discharges. There has been some local scour in the relatively small channels that lead toward the Gulf, but, due to the deterioration of the overbank area, the overall trend has been toward reduction of discharge capacity. This trend has been especially noticeable since 1932 in spite of a large amount of improvement dredging in this area done mostly between 1932 and 1935.

The deterioration of flood-carrying capacity of the floodway below the end of the river levees has reached a point such that to carry the planned project flow of 1,500,000 cfs, improvement work and encouragement of channel development by scour will be required. However, it is not likely that a drastic lowering of stages at the end of the river levees will result. Moreover, backwater computations show that a large change of stage at the downstream end of the river levees would cause only a small fraction of the same amount of change at the head of the Atchafalaya River. Therefore, changes that may occur in the lower basin were omitted from consideration when developing the minimum probable tailwater curves.

Changes that may occur in the leveed reaches and between the leveed reaches and the structures will be the primary causes of changes in tailwater conditions. Channel enlargement is accompanied by reduction of slopes, and channel enlargement is controlled by the bed-sediment carried and by the average total annual water discharge. It is believed that the water-sediment ratio of the discharge through the proposed structures will be about the same as that of the Mississippi River. This will not represent a significant change from the situation that now exists at Old River. This being so the rate and degree of enlargement of the leveed reaches of the Atchafalaya can be affected by regulation of water discharge. However, there are other needs that must be considered in deciding on a plan of regulation. Particularly, the needs of

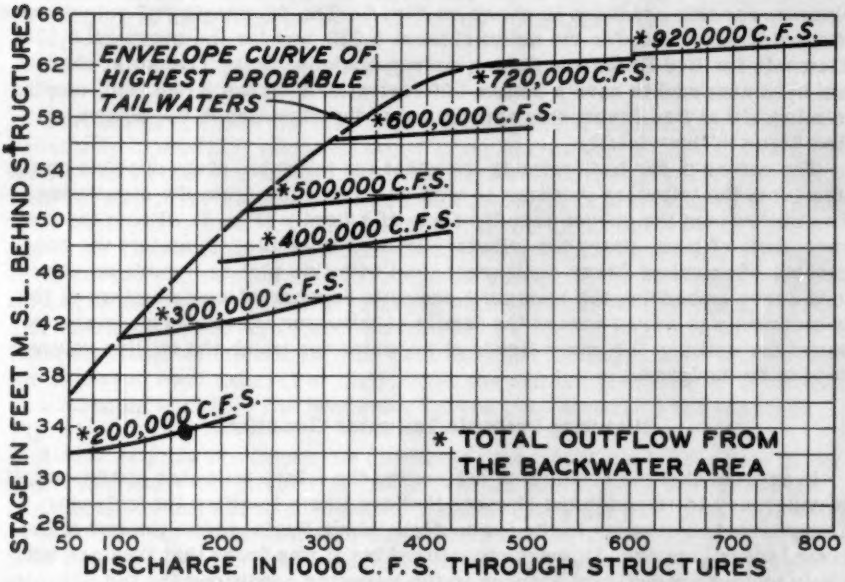

FIGURE 7 - CURVES OF MAXIMUM PROBABLE TAILWATER CONDITIONS

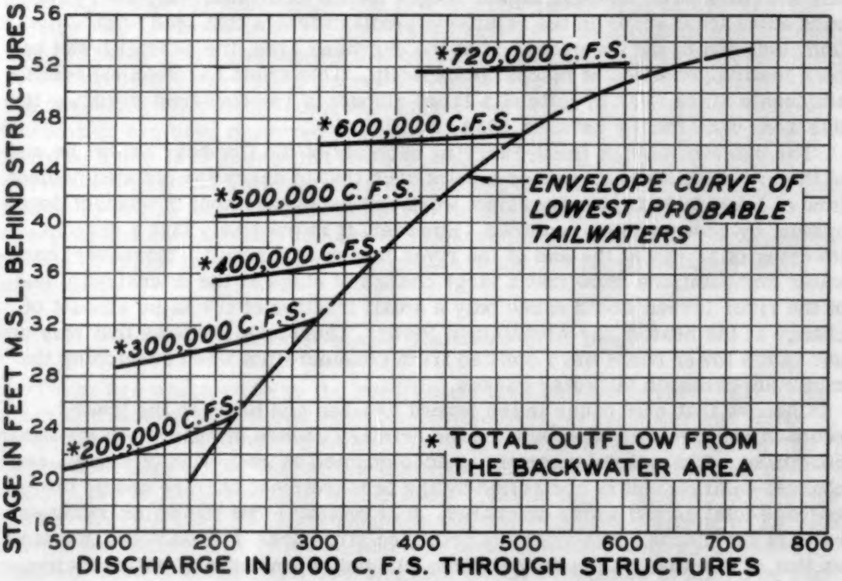

FIGURE 8 - CURVES OF MINIMUM PROBABLE TAILWATER CONDITIONS

flood control may require the Atchafalaya to be allowed to enlarge considerably; because of this and because of the inevitable uncertainty as to what the future needs may be, it is proper to provide against the type of regulation which would produce the minimum tailwater. This means that in order to develop a safe and conservative design, it is better to assume that the Atchafalaya River will be allowed to seek an equilibrium channel size and slope consonant with the average discharges which will occur if only a minimum degree of regulation is exercised.

In the leveed portion of the Atchafalaya River a flattening of slope has been observed for many years, and, as indicated by the foregoing, may be expected to continue. Analytical sediment studies have indicated that for three years during which small to large floods were experienced, the equilibrium slope for the discharges occurring under natural conditions would be about half as steep as the slope observed. The lowering resulting from a reduction to equilibrium slopes was computed to be about 10 feet at high stages. This constitutes an increase in discharge capacity for the Atchafalaya River of about 200,000 cfs for equivalent, high stages. In developing the curves of minimum tailwater the 10-foot lowering was assumed to obtain from project flood stage to bankfull; below bankfull it was tapered out to zero at mean sea level.

Between the head of the Atchafalaya River and the structures much of the flood flow will be overbank, at least for some years after the structures are put in operation. Under these circumstances the slopes will be quite flat. Since the lowering at the head of the Atchafalaya will reduce the depth of overbank flow, thereby tending to increase the slope to the structures, a considerable increase in size of the outlet channel may occur without producing any additional lowering and none has been incorporated in the curves of minimum tailwater.

The family of minimum tailwater curves is shown on Fig. 8. An envelope curve embodying the minimum probable concurrent contributions of the backwater tributaries has been drawn across the family of tailwater curves for the purpose of defining the lower limits of tailwater for which the stilling basins need to be designed. In the routings to be discussed in subsequent paragraphs the envelope curves were not used; the whole family of curves was used in order to take into account the effects of tributary inflows.

Structure Performance at Project Flood

As stated earlier, the project provides against a maximum outflow at the latitude of Old River of 3,000,000 cfs. Synthetic hydrographs of the concurrent inflows of the several streams have been prepared such that the resulting outflow is in accord with the project objective. Routing tables have been prepared for the Mississippi River and for the backwater area, for 1950 conditions and for conditions with the structures in place. With Old River closed, the 1950 conditions headwater and tailwater curves described herein were used. Routings of the project flood, with the control structures wide open (the lock closed) show a maximum discharge through the structures of 700,000 cfs. It is indicated that the structures if wide open will deliver under the project flood conditions roughly 100,000 cfs more to the backwater area than would Old River. This much excess capacity is desirable to make the best use of the available storage during floods just smaller than the project flood, and to provide flexibility of distribution in the event the need arises.

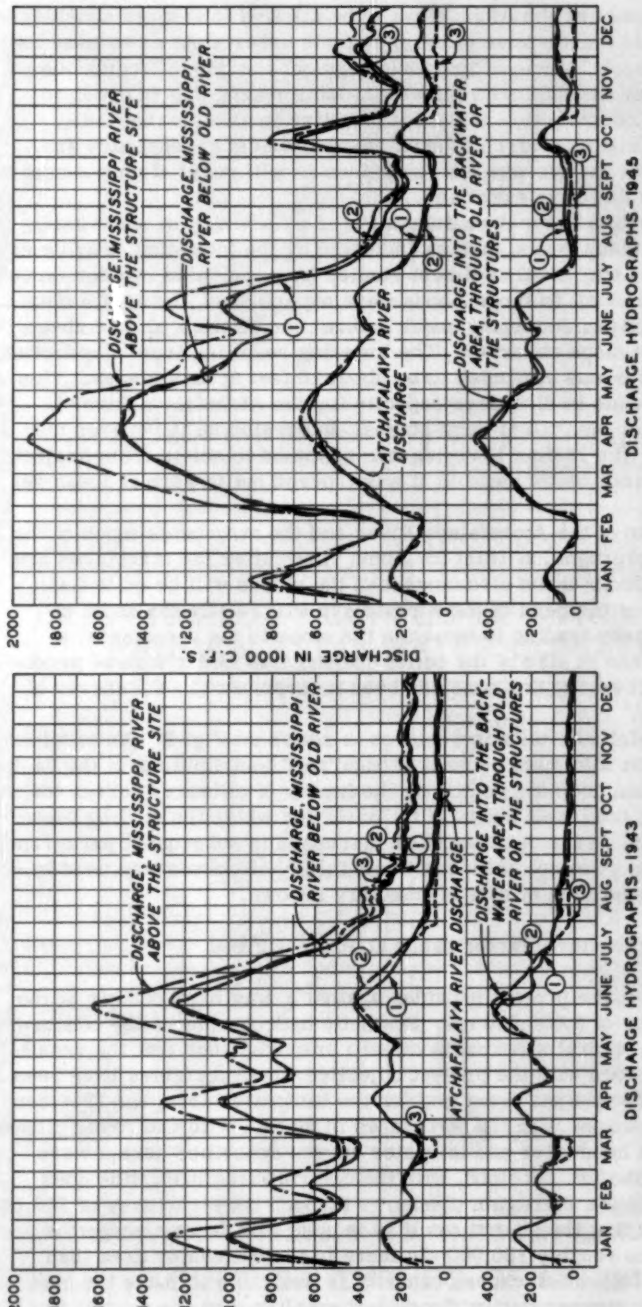

FIGURE 9- YEAR OF A MODERATE FLOOD

FIGURE 10-YEAR OF A LARGE FLOOD

LEGEND

(1) NATURAL CONDITIONS AT TIME OF OCCURRENCE

(2) THE PLAN IN OPERATION, 1950 TAILWATER CONDITIONS, CONTROL STRUCTURES OPERATED AT MAXIMUM CAPACITY

(3) SAME AS (2) EXCEPT FOR A MINOR DEGREE OF REGULATION

Large and Moderate Floods

The plan envisages an overbank structure with crest at 52 feet msl, having 3,200 feet of clear opening, combined with a low-sill structure having 352 feet clear opening with crest at 10 feet msl, and 132 feet of clear opening with crest at minus 5 feet msl. These are the dimensions that have been tentatively determined to be the most advantageous, consistent with economy. For the project flood very similar results can be obtained if the deeper structure is omitted and the overbank structure made longer; and such an arrangement would be less expensive. But such an arrangement would not meet the moderately large flood, medium stage, and low water criteria. To explore the needs for satisfaction of these criteria, routings were made for more moderate floods. The routings are for 1943, a year of moderate flooding which almost forced the operation of Bonnet Carrre Floodway, and for 1945, during which the second largest flood of record in this vicinity was experienced. The results are shown on Figs. 9 and 10. On each figure, hydrographs numbered 1 refer to the Old River open condition, and hydrographs numbered 2 refer to the plan in operation, structures fully open. Hydrographs numbered 1 represent the published discharge which occurred under natural conditions; and in consonance, for hydrographs numbered 2, the condition of the Mississippi River is that of the time of occurrence. An exception to the foregoing statement is that during the flood periods the flows of the Atchafalaya River and of the Mississippi River below Old River were routed for both the open and controlled conditions through tables incorporating 1950 conditions in those rivers. For hydrographs numbered 2, the "maximum probable" tailwater curves were used.

With respect to the large flood criterion, Fig. 10 shows that at the crest of a flood of the magnitude of 1945 the structures will approximate the performance of Old River. For a moderate flood (Fig. 9) the proposed plan is again capable of duplicating the discharge of Old River. The structures can approximate the performance of Old River at about the Mississippi River discharge (1,250,000 cfs below Old River) requiring the operation of one of the floodways. Thus the operation of the plan would not force the operation of either Morganza or Bonnet Carre on occasions when they would not otherwise be required to open.

Medium Stages

With respect to this criterion, the performance of the structures is satisfactory. During medium stages the discharge can be made to approximate the flow which would occur under natural conditions.

Low Water Flow

At extreme low-water stages the flow is dependent on the slot made up of three bays extending to minus 5 feet msl and having a total of 132 feet clear opening. The lowest stage recorded at Red River Landing since 1900 was 4.7 feet mean sea level, 3 November 1939, on which date the measured discharge was 14,000 cubic feet per second through Old River and 19,000 in the Atchafalaya River at Simmesport. For the same circumstances repeated, but with the structures in place and 1950 conditions in the Atchafalaya River channel, the indicated discharge through the slot is 11,000 cubic feet per

second and the Atchafalaya River discharge 16,000 cubic feet per second.

Depending on the degree of submergence by tailwater, the slot could discharge from 15,000 to 22,000 cubic feet per second at the stage at which the 10-foot mean sea level crest is overtopped, the latter figure being for 1950 tailwater conditions and no flow from the backwater tributaries. Ten feet mean sea level is approximately mean low water at this location.

In this connection, it is to be noted that tailwater stages are expected to be lowered by the enlargement of the outlet channel and by the further enlargement of the Atchafalaya River, as explained in an earlier paragraph, increasing the low-water discharge capacity.

Based on records of the last 55 years, there are on the average only 20 days per year during which the water surface at the structure would be below the ten-foot, msl, sill. There will be even less time in future years as Mississippi River flow is supplemented by additional upstream reservoirs. Multiple purpose reservoirs, constructed in the Missouri and Ohio River basins have increased substantially the low-water flow of the Lower Mississippi River, effecting an increase for both the 1953 and 1954 low water periods of more than 30,000 cfs. These additions to Mississippi River flow tend to increase the amount of water that can be diverted into the Atchafalaya. Low water flow from the backwater tributaries will also be increased by the operation of multiple-purpose reservoirs constructed in the basin. The average minimum flow at Alexandria (on Red River, see Figure 1) for the ten-year period preceeding 1944, when Denison Dam was placed in operation, was 2550 cfs, and for the following ten year period was 3960 cfs. An increase in Ouachita River low water flow of about 1,000 cfs is expected with completion of Blakely Mountain Dam in the near future. It will also be possible to divert 1,000 to 2,000 cfs through the proposed lock at Old River to supplement low flows, if this should prove to be desirable.

Effects of Minimum Tailwater

Fig. 11 shows the effects of the minimum tailwater conditions on the 1945 flows repeated with the plan in operation. In this figure the hydrographs of the 1945 flows (Atchafalaya River and Mississippi River below Old River in 1950 condition during the flood period) have been repeated to form a basis for comparison. Hydrographs 2A show for structures having approximately the hydraulic characteristics of the presently proposed plan, the effects on flows that would result from the minimum tailwater conditions. In these routings the structures were kept fully open. The hydrographs show that a major part of the increased capacity of the Atchafalaya would be utilized.

Regulation to Limit Enlargement of the Atchafalaya River

The indications are that enlargement of the leveed reaches of the Atchafalaya has occurred mostly during large floods and filling has occurred during moderate stages. Because at medium stages in the leveed reaches the water drops its sediment load there and at the same time produces bankfull stages in the relatively small channels below the end of the levees, it is believed that enlargement of the lower reaches occurs principally during moderate stages. It appears, therefore, that the best type of flow regulation, from the standpoint of favorably affecting the regimen of the Atchafalaya River and reducing the problems of maintenance both in the lower basin and in the outlet

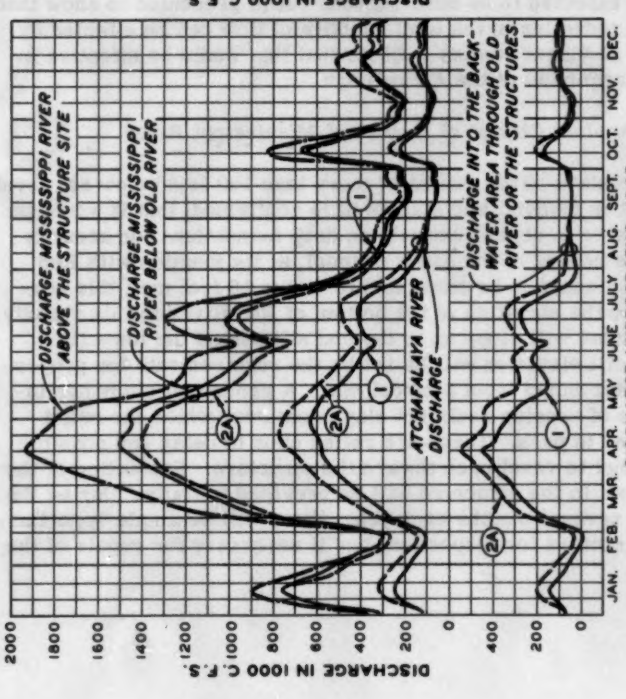

FIGURE 11 - EFFECTS OF MINIMUM TAILWATER CONDITIONS

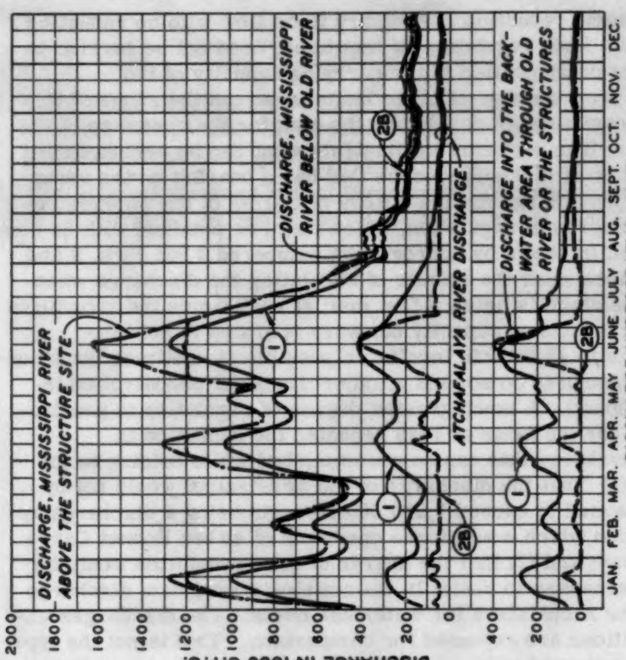

FIGURE 12 - POSSIBILITIES FOR REGULATION OF FLOW

LEGEND

(1) NATURAL CONDITIONS AT TIME OF OCCURRENCE.

(2A) THE PLAN IN OPERATION, MINIMUM TAILWATER CONDITIONS.

(2B) THE PLAN IN OPERATION, 1950 TAILWATER CONDITIONS. A LARGE DEGREE OF REGULATION.

channel consists of some reduction in ordinary flood flow with no reduction in medium stage flow. Such regulation, if instituted, need not be carried to the point of producing deterioration of the carrying capacity of the Atchafalaya. There are a number of other matters beyond the scope of this paper, that will be considered in developing the plan for the operation of the structures. However, it is pertinent to illustrate what can be accomplished in the way of regulation of discharge, since this has a bearing on the assurance the structures provide against the change of course of the Mississippi River. A minimum degree of regulation, which could be obtained with no conflict with other needs, is shown by hydrographs numbered 3 (on Figs. 9 and 10). These hydrographs show the results of regulating the discharge whenever the New Orleans stage is below 8 feet msl; at such times the structures were not operated at maximum capacity but were operated to hold the Atchafalaya discharge to 45,000 cfs insofar as possible (sometimes the structures cannot put in enough to do so, and at other times the uncontrolled inflows exceed that amount). A much greater degree of regulation is practicable. Fig. 12 shows for the flows of 1943 repeated the reduction in Atchafalaya River discharge that can be accomplished while holding to the following criteria: (1) That the discharge of the Mississippi would not by such regulation be caused to exceed 1,200,000 cfs, this being a discharge approaching the condition which would force operation of either Bonnet Carre or Morganza Floodway; and (2) that the degree of such regulation would be reduced whenever necessary to assist in maintaining a minimum discharge of 45,000 cfs down the Atchafalaya for water use needs. The discharges under open river conditions are repeated for comparison. This is not the type of regulation that is expected to be advantageous but is presented to show that a very considerable reduction in the total quantity of flow can be effected by practicable means. Sediment studies indicate that this would be effective in reducing further enlargement of the Atchafalaya.

Assurance Against Change of Course of Mississippi River

Since in the Atchafalaya River depths of more than 100 feet below sea level are quite common, it is evident, and studies have confirmed, that it would be impracticably expensive to depend entirely on deep foundations for security against erosion. The bottom of the outlet channel, at its junction with Red River, is tentatively planned to be excavated to minus 10 feet msl; this corresponds generally to the elevation of the bottom of Red River in this vicinity. From thence the channel will slope up to the end sill behind the low-sill structure. The end sill elevation will be set by the requirements for proper stilling action, as determined from a model test. A geological study has indicated the existence of erosion resistant clay plugs over which the outlet channel will be caused to pass. These are relied upon to delay any possible tendency of the channel to ravel back toward the structure. Protection of the outlet channel adjacent to the structure will be provided initially. During the low-water period of several months each year, there will be ample opportunity to close off the structure for inspection, and if necessary for repair of the outlet channel.

REFERENCES

1. "Old River Diversion Control: The General Problem" by Brigadier General John R. Hardin, President, Mississippi River Commission, Proceedings Paper 906, ASCE, Vol. 82-3.

2. Old River Diversion Control Structures, Soils and Foundation Studies, by W. J. Turnbull, M. ASCE and W. G. Shockley, M. ASCE.

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JOURNAL

WATERWAYS DIVISION

Proceedings of the American Society of Civil Engineers

OLD RIVER DIVERSION CONTROL: FOUNDATION DESIGN

W. J. Turnbull[1] and W. G. Shockley,[2] Members, ASCE
(Proc. Paper 908)

FOREWORD

At the Convention of the Society in St. Louis, Mo., in June, 1955, a luncheon address and four papers presented at a Joint Session of the Waterways, Hydraulics, and Soil Mechanics and Foundations Division formed a sysposium entitled "Old River Diversion Control." Following that meeting, the several papers were reviewed by the appropriate Technical Divisions, revised to meet publication requirements, and are presented herewith under the sponsorship of the Waterways Division.

Contributors of discussion are requested to address themselves to a selected paper in the group. If more than one author is being addressed, separate discussions should be presented.

The papers are grouped under the broad title, "Old River Diversion Control." The first of these papers, "The General Problem," (Proceedings Paper 906) by John R. Hardin, M. ASCE, presents the history of major Mississippi River diversions and analyses observations and studies leading to the conclusion that diversion of the Mississippi River through the Atchafalaya River is imminent; it is concluded that, because of the consequences, this diversion should be prevented. E. A. Graves, A. M. ASCE, in his paper "Hydraulic Requirements" (Proceedings Paper 907), cites the fact that control of the Old River has become necessary because of the threat that the Mississippi River will adopt the course of the Atchafalya River; the choice site was influenced by many conditions among which are the prevailing and prospective tailwater conditions. To design the structures involved in this project, hydraulic models were used extensively according to "Use of Models" (to be published subsequently) by T. E. Murphy, M. ASCE. In "Foundation Design" (Proceedings Paper 908), W. J. Turnbull and W. G. Shockley, Members ASCE, present the general geology and soils condition for the low-sill overburden, and navigation-lock structures proposed for the diversion control.

Note: Discussion open until August 1, 1956. Paper 908 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, No. WW 1, March, 1956.

1. Chief, Soils Div., Waterways Experiment Station, Corps of Engrs., U. S. Dept. of the Army, Vicksburg, Miss.

2. Chief, Embankment and Foundation Branch, Waterways Experiment Station, Corps of Engrs., U. S. Dept. of the Army, Vicksburg, Miss.

"Structures Required" (Proceedings Paper 909) by Norman R. Moore, M. ASCE, is the last of the papers; for the positive control of the flow diversion, structures for flood control and a navigator lock must be provided.

SYNOPSIS

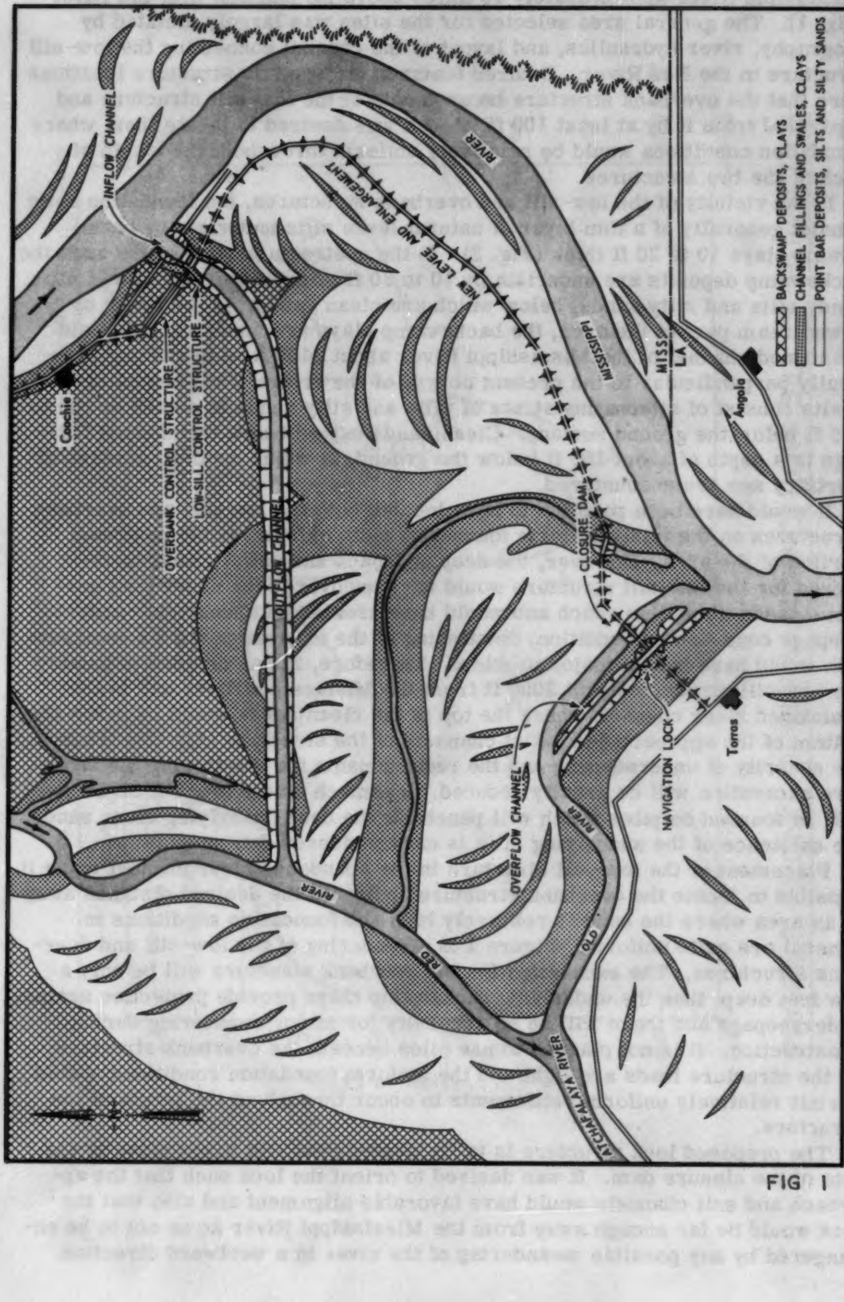
The low-sill, overbank control and navigation lock structures proposed for the diversion control of Old River are founded on a variety of soil types typical of those found in the Lower Mississippi River alluvial valley. Detailed descriptions are given of foundation conditions for the low-sill structure. This structure is a gated weir founded on silty soils about 50 ft below the ground surface. Important considerations in design were excavation dewatering, seepage control, design of pile foundation, and preload fills at the abutments to minimize settlements.

INTRODUCTION

The structures proposed for the control of Old River are the low-sill and overbank control structures and the navigation lock. Figure 1 shows the general locations of the various structures in the proposed improvement plan. This paper discusses the geology and general soils conditions for the three structures, with emphasis on those features which were significant in the final site selection. More detailed descriptions of foundation conditions and design features are given only for the low-sill structure, inasmuch as the design of the other two structures is still in the preliminary stages.

Geology and Site Selection

Before going into the details of subsurface conditions at each structure location it may be helpful to describe briefly the general sequence of sediments found in the lower Mississippi Valley. Practically the entire alluvial valley is underlain by massive sand deposits on the order of 100 to 150 ft thick; these sands are coarse and graveliferous near the base and grade into fine sands near the top of the layer. Superimposed on the underlying sands are alluvial sediments of varying character and thickness, depending upon their mode of formation. These surface materials include uniform clay deposits ranging in thickness from 10 to 60 ft formed by long-time deposition of river sediments in backwater areas and which are termed backswamp clays. In the areas of river meandering, the cutting of river banks and deposition downstream has formed so-called point bar deposits consisting of alternating ridges and swales; in these, top stratum silts and clays are relatively thin overlying sand ridges, and become as deep as 20 to 30 ft in intervening swale areas. River cutoffs result in oxbow lakes which eventually fill with sediments ranging from soft clays to silts and sands, depending on the mode of formation. These filled channels are sometimes as deep as the present river, or between 60 and 150 ft from the ground surface. The pattern of river meandering over a period of many years has resulted in a complex distribution of near-surface soils such that in many areas all of the abovementioned types of deposits may be found within a relatively short distance of one another. Thus, it may be seen that an engineer needs information on the geologic conditions in this region in order to make an intelligent selection



OLD RIVER CONTROL-PLAN OF IMPROVEMENT

FIG 1

of sites for locations of important engineering structures.

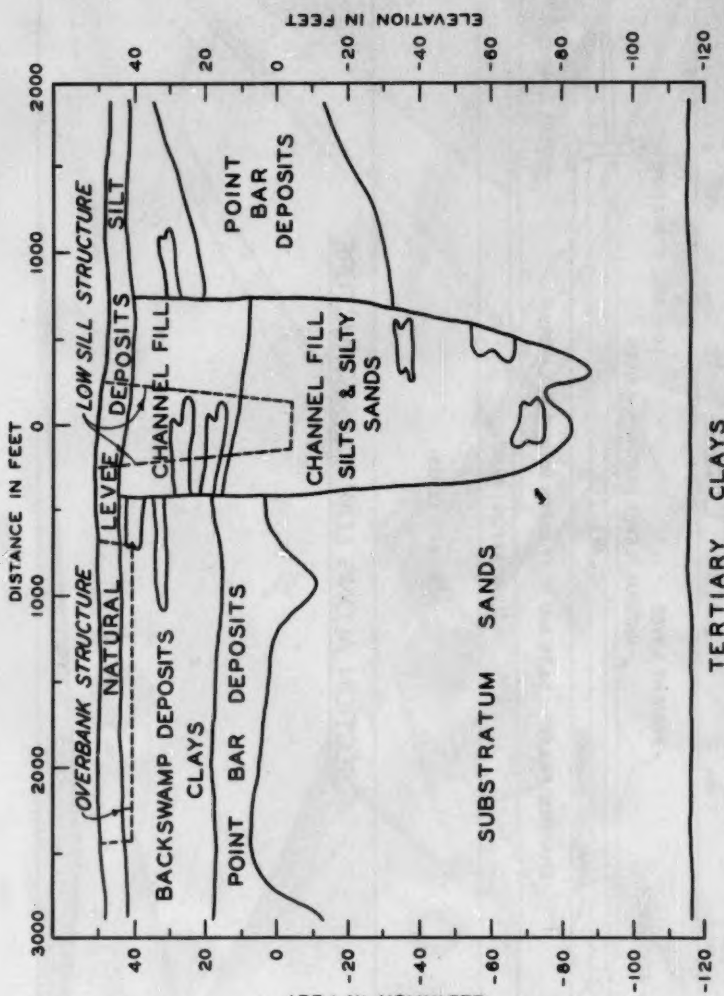
The low-sill and overbank structures are located on the west bank of the Mississippi River approximately 10 miles above its junction with Old River (Fig. 1). The general area selected for the sites was largely dictated by geography, river hydraulics, and layout of the channel connecting the low-sill structure to the Red River. Desired features for specific structure locations were that the overbank structure be upstream of the low-sill structure and separated from it by at least 100 ft; also it was desired to locate them where foundation conditions would be relatively uniform throughout the length of each of the two structures.

In the vicinity of the low-sill and overbank structures, the foundation soils consist generally of a thin layer of natural levee silts underlain by backswamp clays 10 to 20 ft thick (Fig. 2). In the upstream portion of the area the backswamp deposits are underlain by 10 to 30 ft of point bar deposits of silts, sandy silts and silty sands, below which are clean sands. In the lower or downstream part of the area, the backswamp clays are underlain by an old abandoned channel of the Mississippi River about 2400 ft wide running practically perpendicular to the present course of the river. The channel deposits consist of alternating strata of silts and silty sands to depths of 100 to 125 ft below the ground surface. Clean sands extend below the channel fillings to a depth of about 165 ft below the ground, at which depth firm clays of Tertiary age are encountered.

It would have been possible to have located both the low-sill and overbank structures on the fairly uniform foundation soils existing in the upstream portion of the area. However, the deep approach and outlet channels required for the low-sill structure would have uncovered the underlying pervious sands along this reach and would have created an undesirable underseepage condition. In addition, dewatering of the excavation during construction would have been a major problem. Therefore, it was decided to locate the low-sill structure about 2000 ft from the Mississippi River and in the abandoned river channel, where the top of the clean sands would be below the bottom of the approach and outlet channels of the structure (Fig. 3). Thus, the severity of underseepage and the requirements for dewatering the structure excavation will be greatly reduced. Inasmuch as the low-sill structure will be founded on piles which will penetrate the deep underlying clean sands, the existence of the underlying silts is not considered detrimental.

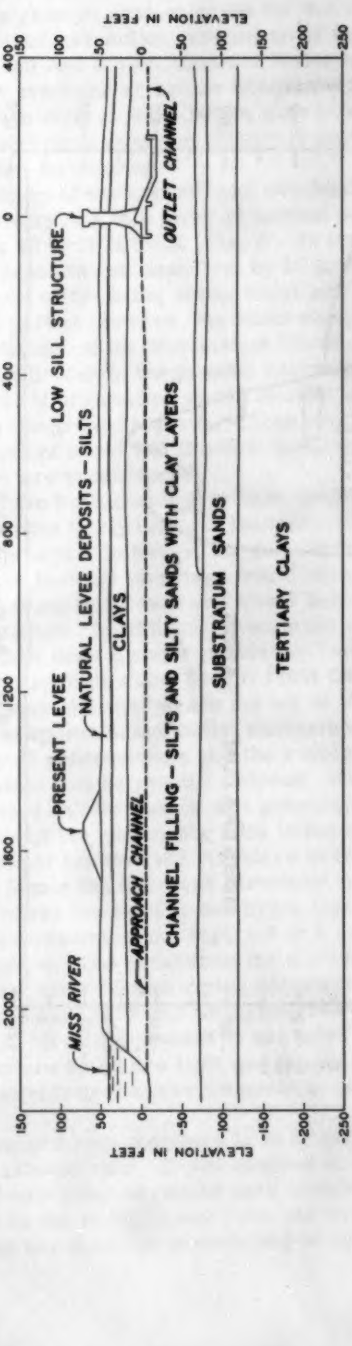
Placement of the low-sill structure in the abandoned river channel made it possible to locate the overbank structure upstream the desired distance away in an area where the sand is relatively high and foundation conditions in general are quite uniform. Figure 4 is a rendering of the low-sill and overbank structures. The excavation for the overbank structure will be only a few feet deep, thus the underlying backswamp clays provide protection against underseepage and there will be no necessity for major dewatering during construction. It is not planned to use piles beneath the overbank structure, as the structure loads are light and the uniform foundation conditions should permit relatively uniform settlements to occur throughout the length of the structure.

The proposed lock structure is to be located near Old River close to the site of the closure dam. It was desired to orient the lock such that the approach and exit channels would have favorable alignment and also that the lock would be far enough away from the Mississippi River so as not to be endangered by any possible meandering of the river in a westward direction.



PROFILE ALONG OVERBANK AND LOW SILL STRUCTURES

FIGURE 2



SECTION ALONG LOW SILL STRUCTURE

Figure 3

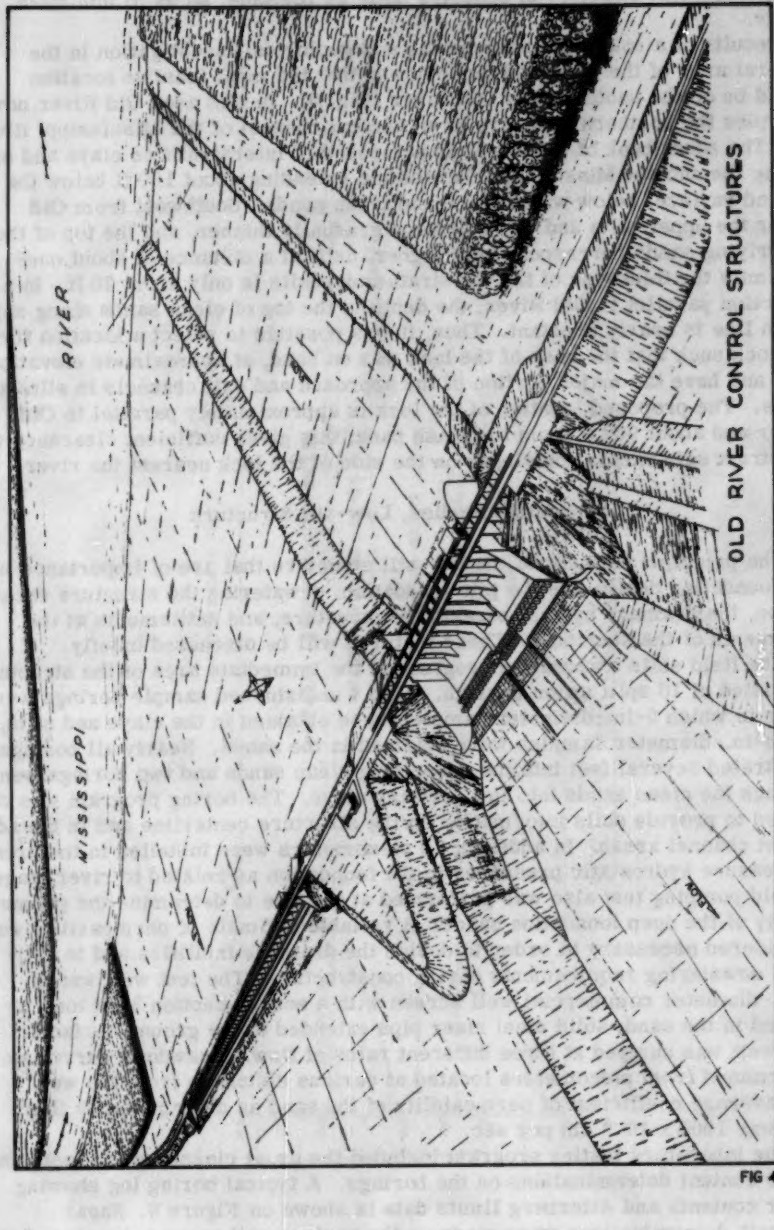


FIG 4

From a foundation standpoint it was desired to have a firm sand foundation beneath the lock, but at the same time have as much of the bottom of the approach and exit channels in cohesive soils as possible, so as to minimize scour.

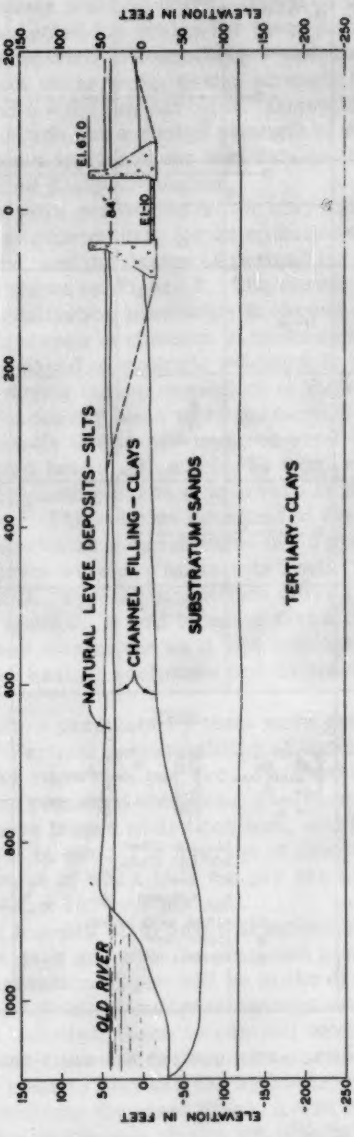
Results of a comprehensive boring and geological investigation in the general area of the proposed lock showed that the most feasible location would be on the south side of Old River (Fig. 5). In this area Old River now occupies the northern boundary of an ancient channel of the Mississippi River. The south bank of Old River is composed of interstratified clays and silts filling the ancient Mississippi channel and extending about 100 ft below the ground surface, below which are found clean sands. Southwest from Old River the upper silts and clays become gradually thinner, and the top of the underlying sands correspondingly higher, until at a distance of about one-half mile the thickness of the top stratum deposits is only about 20 ft. In a direction parallel to Old River, the depth to the top of clean sands along any given line is nearly constant. Thus, it was possible to select a location for the lock such that the base of the lock was on sand, at approximate elevation -15, and have the major portion of the approach and exit channels in silts and clays. The proposed location of the lock is approximately parallel to Old River and about 900 ft from the south bank; this gives sufficient clearance to construct safe excavation slopes on the side of the lock nearest the river.

Foundation Studies, Low-sill Structure

The principal features of the low-sill structure that are of importance in the foundation design are the pile foundation, dewatering the structure excavation, the drainage system beneath the structure, and settlements at the abutments of the structure. These features will be discussed briefly.

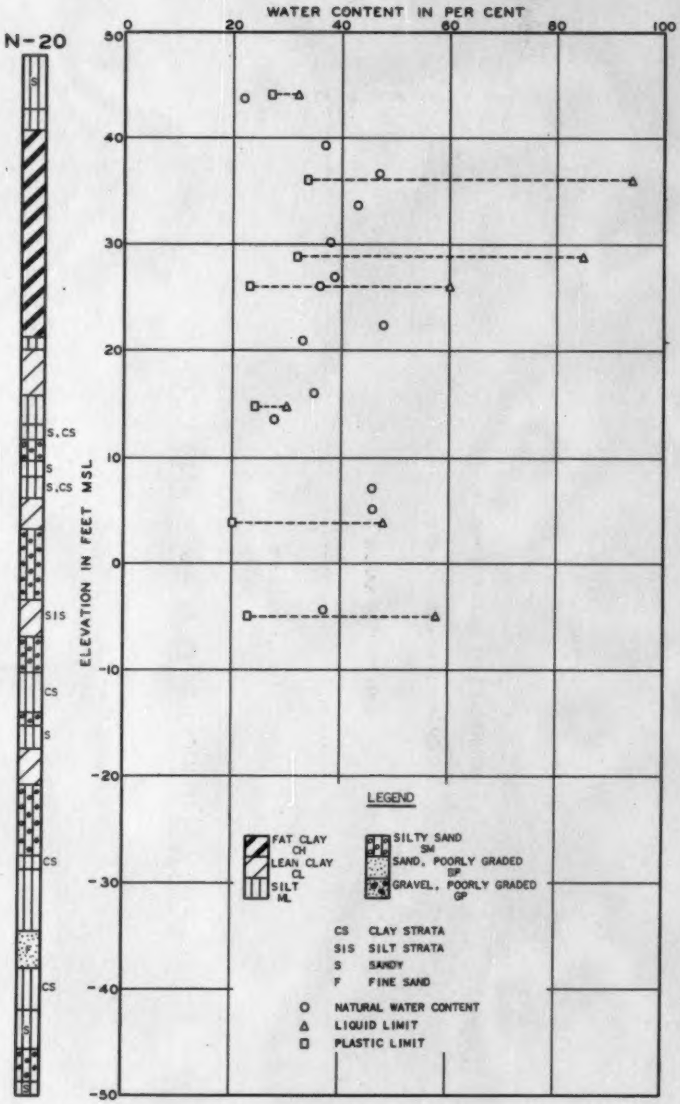
The field soils exploration program in the immediate area of the structure consisted of 10 split spoon borings. Also 7 undisturbed sample borings were made in which 5-in.-diameter samples were obtained in the clays and silts, and 3-in.-diameter samples were obtained in the sands. Nearly all borings penetrated several feet into the underlying clean sands and two borings went through the clean sands into the Tertiary clays. The boring program was designed to provide soils information on the structure centerline and in the adjacent channel areas. In addition, 10 piezometers were installed in the area to measure hydrostatic pressures in the foundation as related to river stage. A field pumping test also was performed at the site to determine the permeability of the deep foundation sands. A reliable estimate of permeability was considered necessary in order to design the drainage facilities and to estimate dewatering requirements during construction. The test well was an 8-in.-diameter commercial well screen with a screen section 30 ft long installed in the sand; solid steel riser pipe extended to the ground surface. The well was pumped at three different rates of flow. Drawdown curves were determined from piezometers located at various distances from the well. The average coefficient of permeability of the sand as determined by this test was 1000×10^{-4} cm per sec.

The laboratory testing program included the usual classification tests and water content determinations on the borings. A typical boring log showing water contents and Atterberg limits data is shown on Figure 6. Shear strength determinations were made on the various soil types encountered at the site using test procedures which were considered appropriate for the soil



SECTION THROUGH OLD RIVER AND LOCK

FIG 5



TYPICAL WATER CONTENT AND LIMIT DATA

Figure 6

types and for loading conditions anticipated during construction and operation of the structure. Shear strength tests consisted of unconfined compression, unconsolidated-undrained and consolidated-undrained triaxial and consolidated-drained direct shear tests on clays. Based on these data a strength of 0.3 ton per sq ft was selected for design for these soils. Silts, sandy silts, and silty sands were subjected to consolidated-undrained triaxial and consolidated-drained direct shear tests; design strength for these materials was selected as $\phi = 28^{o}$ and c = 0.1 ton per sq ft. Consolidated-drained triaxial tests were run on the sands and a design strength of $\phi = 33^{o}$ and c = 0 was selected. Figure 7 shows typical shear test data on the various soil types and their relation to selected design strengths.

Consolidation tests were performed on the clay soils, the filty soils and the clean sand, so as to provide data for computations for settlement of the abutment, the structure, and the design of preload fills. Typical pressure-void ratio curves are shown on Figure 8. The curves for the backswamp clays indicated preconsolidation pressures in excess of normal overburden pressures. This phenomenon is common in backswamp clays of the lower Mississippi valley and based on geologic evidence is believed the result of alternate wetting and drying during deposition of the sediments.

Natural and relative density tests were performed on the sands. Maximum and minimum density tests were made on oven-dry samples by pouring the sand from a constant height into a 2-in. by 4-in. mold for the minimum density test and by compacting the sand in layers in the same mold for the maximum density test.[3] The relative densities of the deep sands ranged from 55 to 96 per cent with an average value of 69 per cent.

Also, compaction tests were run on selected soil types for use in levees, cofferdams, and backfill. Typical compaction curves on representative soil types are shown on Figure 9. It will be noted that a light compactive effort was used on the silt and clay mixes as it was anticipated that these materials would be compacted by hauling equipment and by tractors rather than by rollers.

In addition, laboratory permeability tests were performed on representative silts and sands. Vertical permeabilities of undisturbed silt samples ranged from 0.26 to 34 x 10^{-4} cm per sec. Permeabilities of the sands were determined by tests on remolded samples. The sand samples were washed before testing to remove traces of drilling mud, and test values were corrected to the void ratio in situ. The average of laboratory permeability tests on the sands gave a value of 500 x 10^{-4} cm per sec as compared to a horizontal permeability of 1000 x 10^{-4} cm per sec.

Excavation for the low-sill structure will extend about 50 to 65 ft below the present ground surface and with the proposed guide levees and cofferdams the ultimate excavation slopes will be in the order of 70 to 80 ft high. Based on the results of stability analyses average excavation slopes of 1 on 4 to 1 on 5 have been selected; these slopes will have factors of safety ranging from 1.25 to 1.5. Excavation and construction are to be carried on in the dry. As the natural water table is high and the hydrostatic pressure in the underlying sands directly reflects the river stage, it will be necessary not only to dewater the excavation on the side slopes but also to reduce the hydrostatic pressure in the deep sands in order to prevent blowup of the bottom of the

3. Waterways Experiment Station, "Summary Report of Soils Studies, Potamology Report 12-2, October 1952.

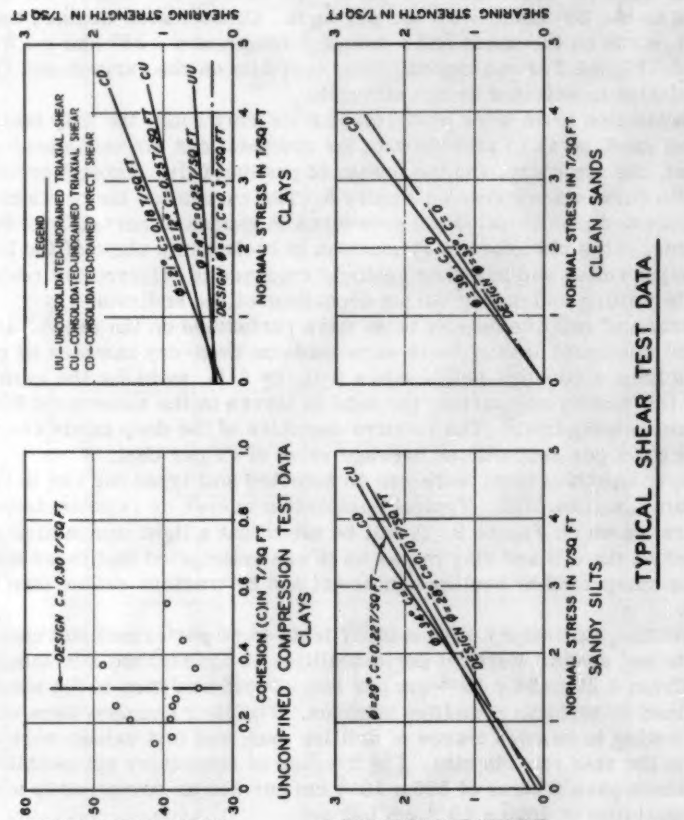
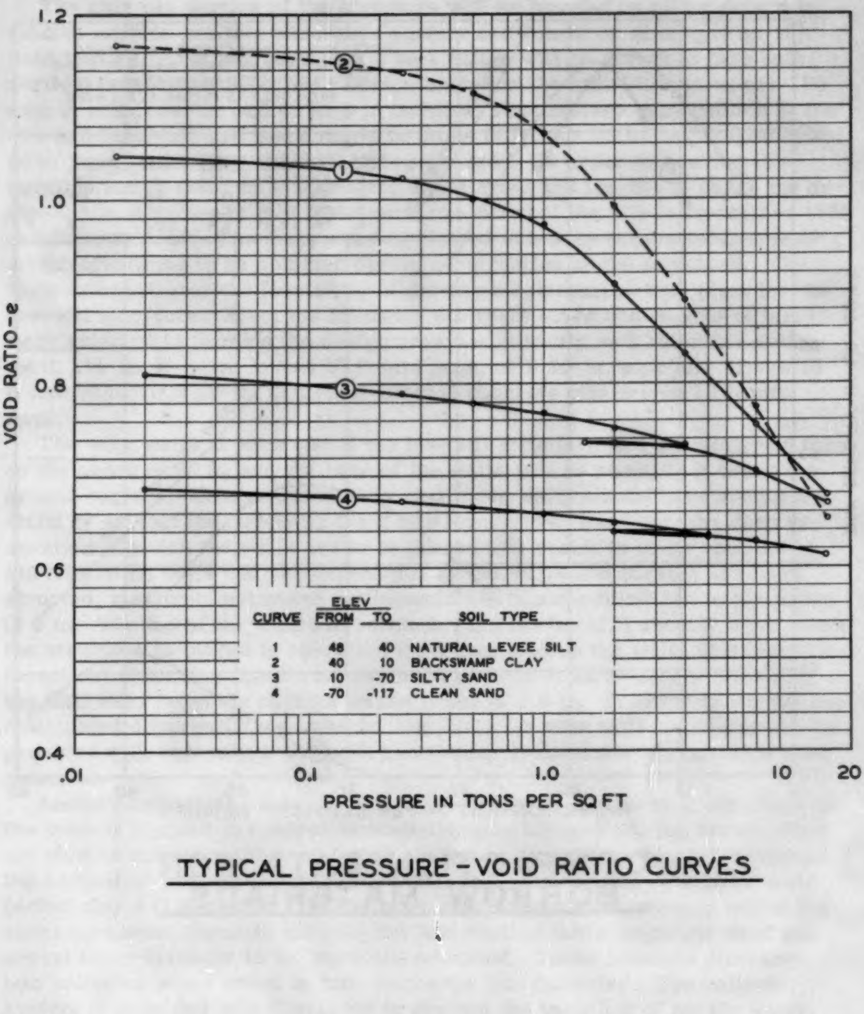
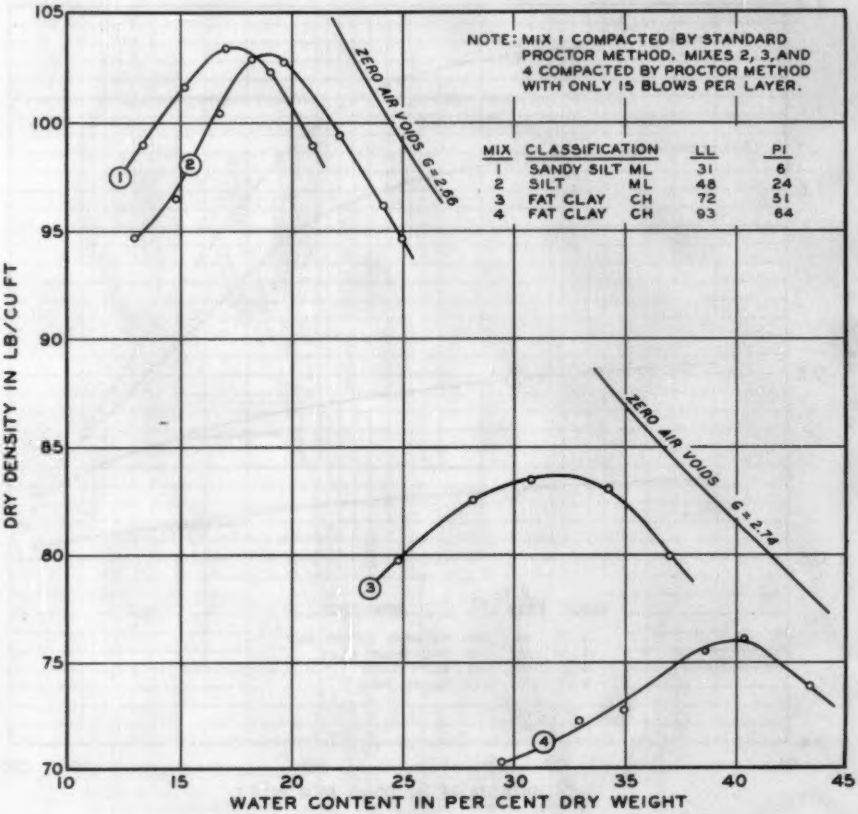

LEGEND
UU—UNCONSOLIDATED-UNDRAINED TRIAXIAL SHEAR
CU—CONSOLIDATED-UNDRAINED TRIAXIAL SHEAR
CD—CONSOLIDATED-DRAINED DIRECT SHEAR
WATER CONTENT IN PER CENT
COHESION (C) IN T/SQ FT
DESIGN C=0.30 T/SQ FT
UNCONFINED COMPRESSION TEST DATA
CLAYS
SHEARING STRENGTH IN T/SQ FT
NORMAL STRESS IN T/SQ FT
CLAYS
CD
CU
UU
DESIGN
SANDY SILTS
CLEAN SANDS
TYPICAL SHEAR TEST DATA

Figure 7



TYPICAL PRESSURE-VOID RATIO CURVES

Figure 8



COMPACTION DATA FOR BORROW MATERIALS

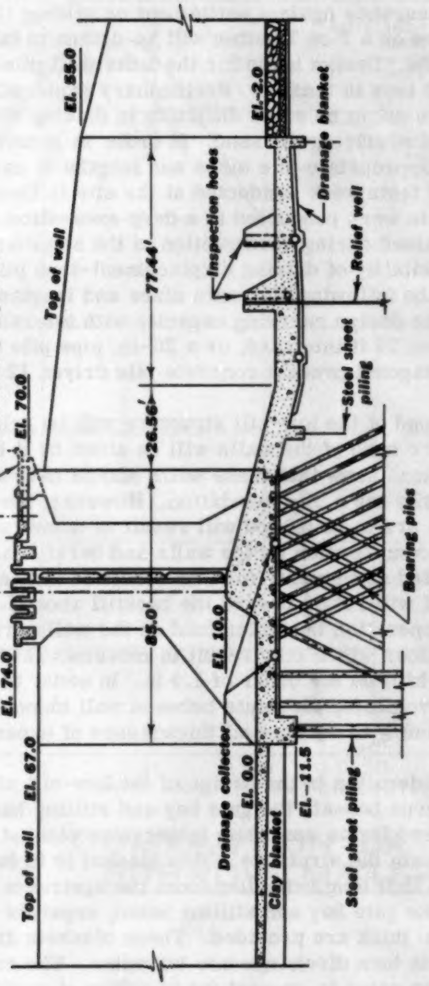
Figure 9

excavation. It is contemplated that a multistage wellpoint system combined with deep wells will be required to accomplish this objective. In addition, excavation slopes will be seeded and ditched in order to control the inflow of surface water from rains.

The gate bay section of the structure will be founded on piling driven to sand to provide positive assurance against settlement or sliding (Fig. 10). Both vertical piles and piles on a 2 on 1 batter will be driven to take both vertical and horizontal loads. Design loads for the individual piles are 100 tons in compression and 40 tons in tension. Preliminary explorations at the site had indicated that there might be some difficulty in driving displacement piles through the silty soils overlying the sand. In order to resolve this question and to determine appropriate pile sizes and lengths to carry the design loads, a series of pile tests were conducted at the site in December 1954 and January 1955. The tests were conducted in a deep excavation simulating actual conditions to be obtained during construction of the structure. The tests demonstrated the feasibility of driving displacement-type piles for the low-sill structure. Also, the following alternate sizes and lengths of piles were selected to provide the design carrying capacity with tolerable settlement: 14-in. H-beam driven 27 ft into sand, or a 20-in. pipe pile driven 15 ft into sand, or a 20-in. octagonal precast concrete pile driven 12 ft into sand.

The wing walls at each end of the low-sill structure will be relatively high, on the order of 60 ft, and the base of the walls will be about 57 ft below the ground surface. Computations show that these walls should have adequate stability without the necessity for a pile foundation. However, the deep excavation in which the walls are to be placed will result in some rebound of the foundation soils and recompression as the walls and backfill are constructed; maximum computed settlements for this condition are in the order of 2 in. The top of the wall will rotate toward the backfill about 1.5 in. When the structure is placed in operation the water load on the walls creates a different set of loading conditions which may result in movement at the top of the wall away from the backfill on the order of 1.0 in. In order to permit the anticipated differential movements, the joints between wall monoliths will be provided with waterstops and with substantial thicknesses of expansion joint material.

Another important consideration in the design of the low-sill structure is the control of uplift pressures beneath the gate bay and stilling basin. Flow net studies indicated the need for an upstream impervious blanket to reduce the amount of seepage beneath the structure. This blanket is to be of compacted clay 4 ft thick and 175 ft long extending from the upstream end of the concrete apron. Beneath the gate bay and stilling basin, separate sand and gravel filter blankets 14 in. thick are provided. These blankets discharge into collector pipes which in turn discharge into tailwater. The collector system is provided with flap gates to prevent the backflow of muddy water, and with manholes and other facilities to permit access for cleaning. In addition, provision had to be made for control of uplift pressures in the deep sands beneath the stilling basin. This has been accomplished by 7 relief wells, 8 in. in diameter, surrounded with a gravel filter, which penetrate into the sands and discharge by free flow into the drainage collector system. These wells are to be installed during initial stages of construction so as to provide assistance in reducing uplift pressures during construction operations. Steel sheet piling has been provided upstream of the gate bay and



TYPICAL SECTION—LOW SILL STRUCTURE

Figure 10

PRELOAD FILLS AND PILE TEST EXCAVATION

Figure 11

between the two drainage blankets to minimize erosion and piping beneath the structure.

The foundation soils at the abutments of the structure will be subjected to significant settlements beneath the adjoining levee fills. Settlement of the foundation will result in a downward drag on the piles which possibly could greatly overload the piles beneath the abutments piers. In addition, settlement at the abutments could cause the earth fill to pull away from curtain walls in this area and thus create potentially hazardous conditions with respect to seepage during high water. In order to minimize these effects, preload fills have been constructed at each end of the low-sill structure so as to reduce settlements to tolerable amounts in advance of construction (Fig. 11). Predicted settlements at the ends of the structure were on the order of 10 to 17 in. and the preload fills were so designed and overbuilt that approximately this amount of settlement would take place in a 12-month period. The preload fills were so constructed that a substantial portion of each will remain in place as part of the permanent construction.

JOURNAL
WATERWAYS DIVISION
Proceedings of the American Society of Civil Engineers

OLD RIVER DIVERSION CONTROL: STRUCTURES REQUIRED

Norman R. Moore,[1] M. ASCE
(Proc. Paper 909)

FOREWORD

At the Convention of the Society in St. Louis, Mo., in June, 1955, a luncheon address and four papers presented at a Joint Session of the Waterways, Hydraulics, and Soil Mechanics and Foundations Division formed a sysposium entitled "Old River Diversion Control." Following that meeting, the several papers were reviewed by the appropriate Technical Divisions, revised to meet publication requirements, and are presented herewith under the sponsorship of the Waterways Division.

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Note: Discussion open until August 1, 1956. Paper 909 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, No. WW 1, March, 1956.

1. Chief, Eng. Div., Mississippi River Commission, Vicksburg, Miss.

SYNOPSIS

Positive control of diversion of flow from the Mississippi River into the Atchafalaya Basin through Old River requires provision of structures that will permit retention of the adopted flood control plan. This stipulation dictates a maximum discharge capacity of not less than 700,000 cfs for the project flood stage. It is also necessary to provide sufficient capacity at lesser stages to make increased frequency of operation of the downstream Morganza and Bonnet Carre floodways unnecessary. For medium stages, the structures must have capacity equivalent to the natural diversion ability of Old River to facilitate the development of channels through the lower Atchafalaya Basin where such flows will result in bankfull stages. To meet the requirements of navigation, a waterway traversing the closure dam must be provided. These requirements are fulfilled in the design of the low-sill and overbank structures, described herein, and in the appurtenant channels, levees, closure dam, and navigation lock for which planning is under way.

General Description

The project area is located on the west bank of the Mississippi River between miles 301 and 313 above Head of Passes, Louisiana. (Figure 1) The control structures will be located between miles 312 and 313, and the navigation lock will be in a land cut south of the mouth of Old River.

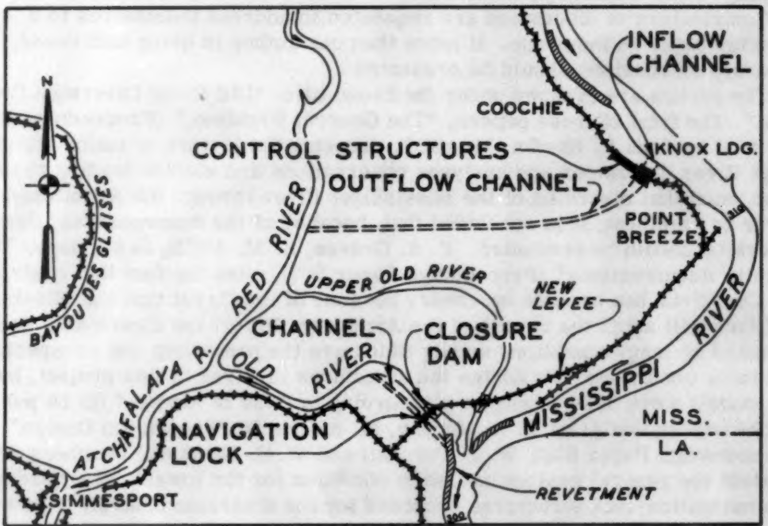

FIGURE 1- PLAN OF PROJECT SITE

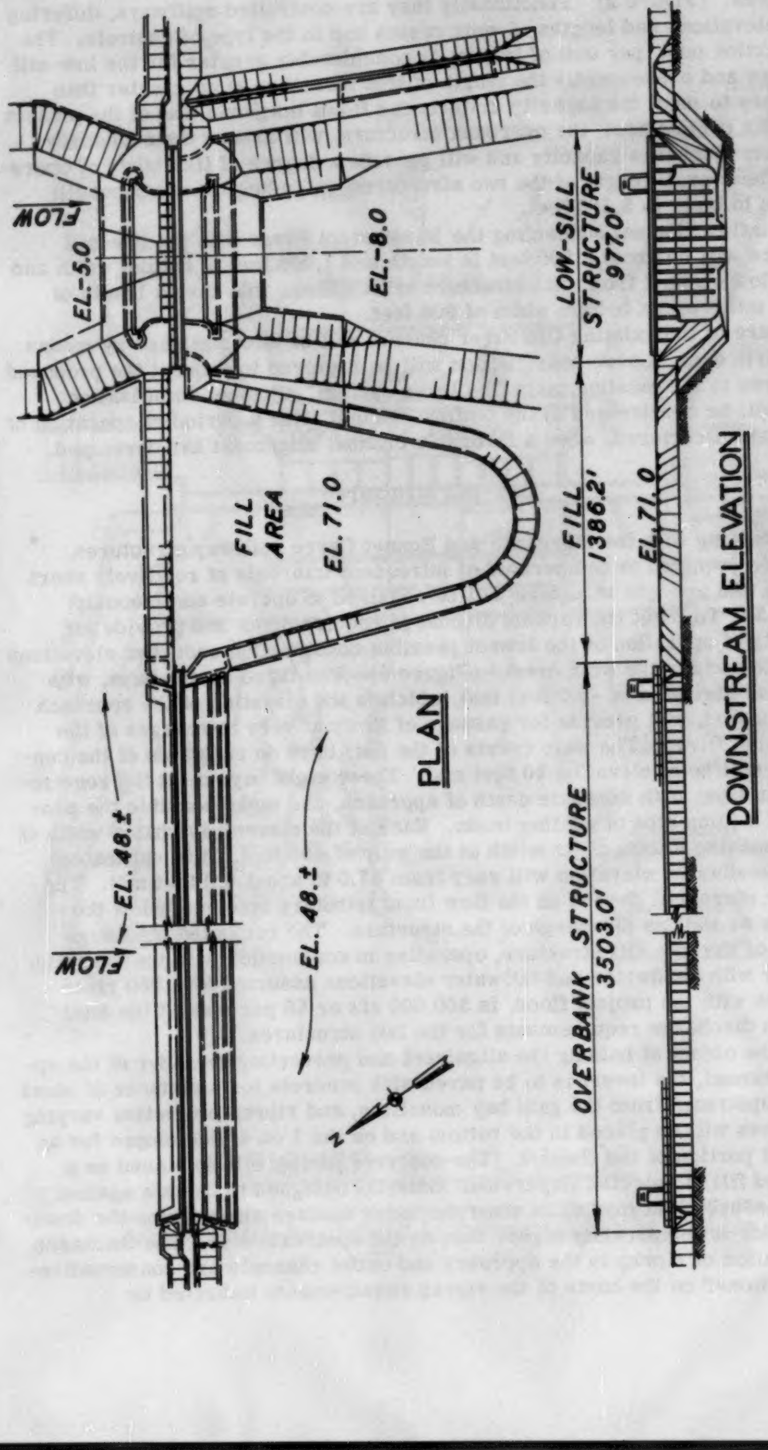

FIGURE 2 - CONTROL STRUCTURES

Considerations of economy led to the decision to provide two separate control structures. (Figure 2) Functionally they are controlled spillways, differing in the elevations and lengths of weir crests and in the type of controls. The construction cost, per unit of length, is considerably greater for the low-sill structure and consequently the length of this structure is no greater than necessary to meet the capacity criteria for flows less than that of the project flood. Its complement, the overbank structure, will assure the necessary maximum discharge capacity and will provide a degree of flexibility of operation. The overall length of the two structures, including the abutment fill common to both, is 5,868 feet.

The inflow channel connecting the Mississippi River with the low-sill structure will be about 2,200 feet in length and 1,000 feet in bottom width and the outflow channel from that structure to Red River will have a length of about 7 miles and a bottom width of 900 feet.

Closure of the existing Old River channel will be accomplished by means of an earth dam. Levee construction will be required to connect the proposed structures to the existing main line levee system. Channel stabilization works will be constructed in the outflow channel after a period of operation of the control structures, when a favorable channel alignment has developed.

Low-Sill Structure

Contrasting with the Morganza and Bonnet Carre spillway structures, which are required to be operated at infrequent intervals of relatively short duration, the low-sill structure will be required to operate continuously. (Figure 3) To meet the various discharge requirements and provide for flexibility of operation at the lowest possible construction cost, two elevations were selected for the weir crest. (Figure 4) The three center bays, with weir crest elevation of -5.0 feet msl, which is the elevation of the approach channel invert, will provide for passage of flows at very low stages of the Mississippi River. The weir crests of the four bays on each side of the center bays will be at elevation 10 feet msl. These eight bays meet the requirements for flow, with adequate depth of approach, and make possible the provision of a jump type of stilling basin. Each of the eleven bays has a width of 44 feet, making a total clear width at the weir of 484 feet. It is estimated that the headwater elevation will vary from 67.0 to about 5.0 feet msl. The tailwater stage will depend on the flow from tributary streams below the structure as well as discharge of the structure. The computed discharge capacity of the low-sill structure, operating in conjunction with the overbank structure with headwater and tailwater elevations assumed for 1950 river conditions with the project flood, is 350,000 cfs or 50 per cent of the total minimum discharge requirements for the two structures.

With the object of holding the alignment and preventing meander of the approach channel, the invert is to be paved with concrete for a distance of about 100 feet upstream from the gate bay monoliths, and riprap protection varying in thickness will be placed in the bottom and on the 1 on 4 side slopes for an additional portion of the channel. The concrete paving will be placed on a compacted fill of selected impervious material designed to be safe against uplift pressures that may exist when the water surface elevation on the downstream side is temporarily higher than on the upstream side. The thickness and gradation of riprap in the approach and outlet channels are conservatively proportioned on the basis of the riprap requirements indicated by

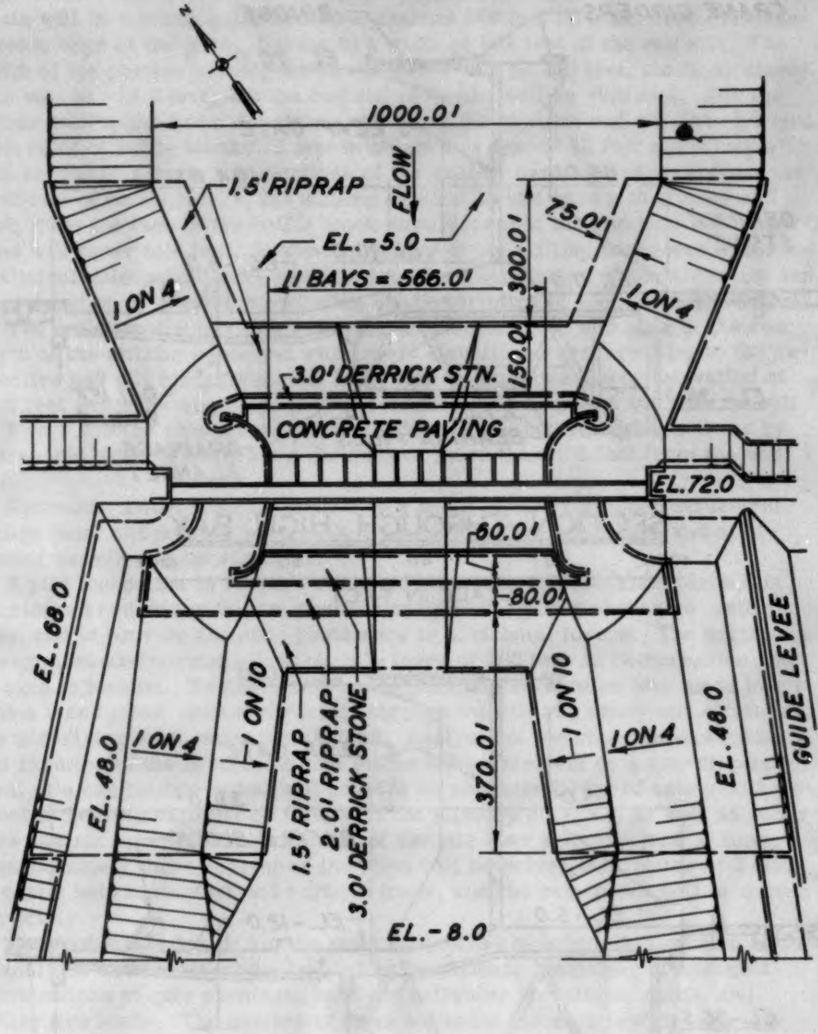

FIGURE 3 - PLAN OF LOW-SILL CONTROL STRUCTURE
NOT TO SCALE

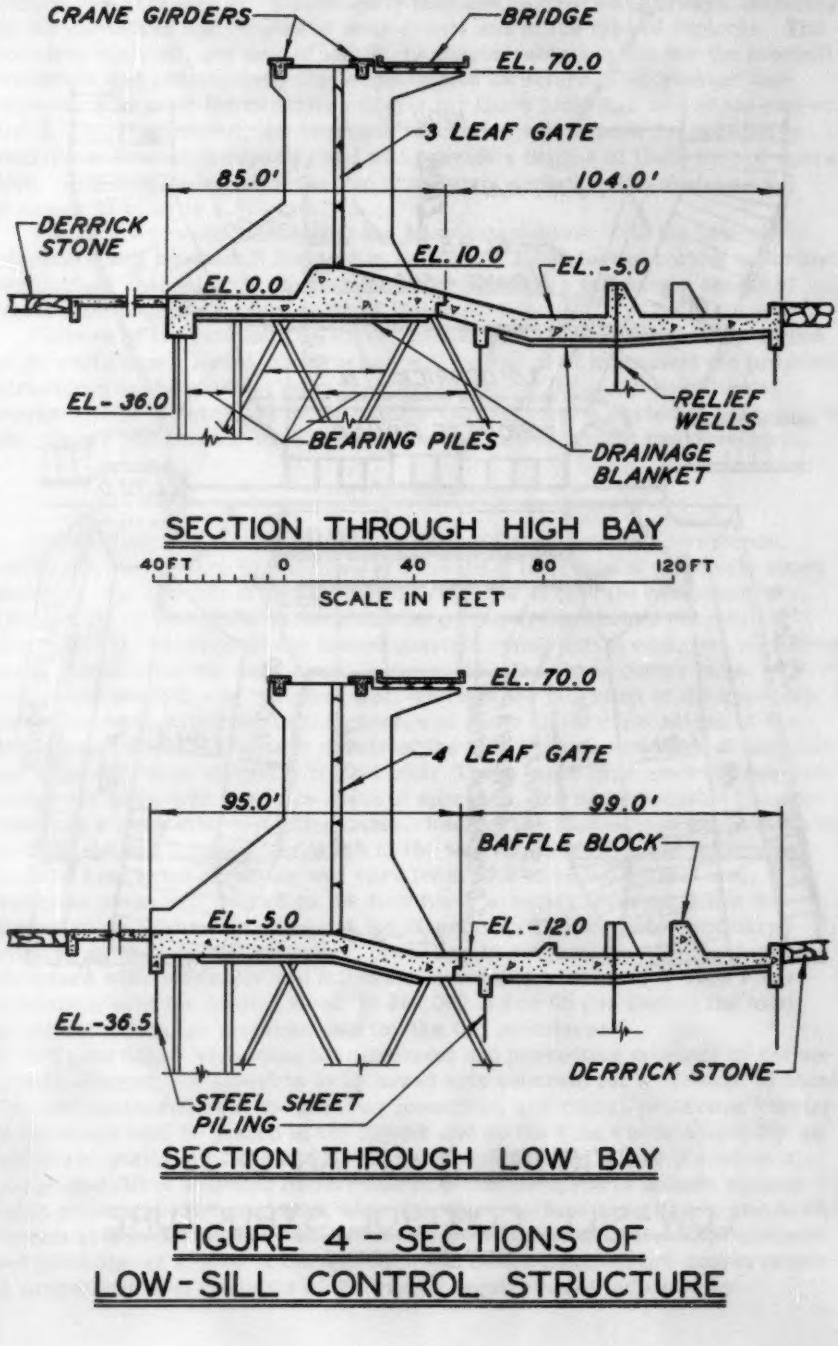

FIGURE 4 - SECTIONS OF LOW-SILL CONTROL STRUCTURE

experiment, taking into account computed average velocities. The stilling basin will be a rectangular concrete channel 566 feet in width near the downstream edge of the piers, flaring to a width of 592 feet at the end sill. The width of the portion serving the lower weirs will be 150 feet, the floor elevation will be -12.0 feet, and the end sill elevation will be -9.0 feet. For the higher weirs, the floor elevations will be -5.0 feet, with end sills at -2.0 feet. Two rows of baffle blocks 10 feet in height and spaced 12 feet apart laterally will be placed across both portions of the stilling basin. Upstream from the first row of baffle blocks, the stilling basin slab will have a thickness of 7 feet; from the face of the baffle block downstream to the end sill, the thickness will taper to 4 feet. Hydraulic design of the stilling basin was based on analytical determinations supplemented by model studies of the structure and consideration of prototype operating characteristics of similar structures.

The outlet channel, with a rectangular section at the end sills of the two parts of the stilling basin and with invert elevations corresponding to the respective end sill elevations, will transition to a uniform invert elevation of -8.0 feet and a bottom width of 900 feet. The side slopes of the channel will be 1 on 4 and the channel bottom and sides will be protected from scour by riprap and derrick stone for a distance of more than 600 feet from the end sill.

Hydraulic model tests on models to a scale of 1:36 furnished structural design data, and pointed out means of improving flow conditions and of effecting economy in construction.

A pile foundation to support the gated structure and abutment piers was considered necessary to give positive assurance against excessive settlement and to provide adequate resistance to horizontal forces. The foundation design assumed normal allowable pile loads of 100 tons in compression and 40 tons in tension. To check this assumption, an excavation was made to elevation 0 and piles, driven vertically through the silt and sandy silt strata into the underlying sand, were test-loaded. Analysis of the observations made has resulted in the conclusion that either a displacement or a non-displacement pile can be driven and will provide an adequate factor of safety with respect to bearing capacity or failure of the piles by plunging, as well as assurance against appreciable settlement of the pile over a long period of time. Approximately three-fourths of the piles will be driven on a batter of 2 on 1 to resist both horizontal and vertical loads, and the remainder will be driven vertically.

The foundation design for the main structure employed the Culmann method of analysis and considered nine loading conditions, consisting of assumed combinations of gate openings, head and tailwater elevations, uplift, and bridge live loads. The number of piles indicated for each design case was calculated for the battered upstream, battered downstream, and vertical piles. The design makes provision for not less than the highest number in each group.

Besides carrying the design loads in compression and tension, the piles must withstand handling and driving stresses. The requirement that the piles must be driven into the "bearing sand," which is defined as the sand in which the driving resistances equal or exceed stipulated values, will result in piles varying in average length from 84 to 98 feet, depending upon whether pipe piles, precast reinforced concrete, or steel H-beam piles are used. Experience at the Morganza Spillway in driving 20-inch octagonal and square precast reinforced concrete piles on a batter of 2:1 in lengths of 100 feet or

greater, as well as driving experience on other projects, furnishes assurance that somewhat shorter piles on the same batter can be satisfactorily driven at this site.

The 44-foot width of gate bays was selected on the basis of an economic study which involved the plotting of curves of the total costs of variable items, such as piers, foundation, gates, and gantry crane against the clear span between piers. The gated portion of the structure is composed of a series of reinforced concrete monoliths consisting of a base supporting two piers tied together at the top by the crane bridge girders, and will be divided by joints in the base at the center of alternate bays into a low flow monolith, transition monoliths, weir section monoliths, and end monoliths. Provision is made of three steel sheet pile cutoff walls. One at the upstream end of the gate bay monolith and one in the cutoff key at the end of the stilling basin will be provided as protection against piping and scour, and an additional row at the upstream end of the stilling basin slab will separate the two drainage blankets. Abutments consisting of bulkhead walls varying in height and supported by a series of piers will connect the gated structure to the adjacent levee at each side. The bulkhead walls will rest in slots in the piers that are so designed as to permit minor movements due to temperature changes and differential settlement. Design of the upstream and downstream wing walls, provided to guide and facilitate the flow of water to and from the weirs, has been aided by the model studies which have furnished data on the most favorable alignment and maximum wave heights. Access across the structure will be provided by a two-lane concrete deck bridge.

Vertical lift gates of the fixed wheel, welded structural steel type will be provided in each of the eleven gate bays to regulate flow through the low-sill structure. Each gate is divided into sections to facilitate handling. The three center gates will have three leaves, each 19 feet in height, and one of 15 feet, with an overall height of 72 feet. The remaining eight gates will have only the three 19-foot leaves, making an overall height of 57 feet. The gates will be provided with rubber seals mounted in the upstream face along the sides and between the leaves. Along the sides, the seals will bear on stainless steel plates anchored in the masonry. At the lower edge of the gate, adjustable corrosion resistant plates bolted to the skin plate will make metal-to-metal contact with the sill.

The operating plan for the structure contemplates that the three center gate bays will be opened first, followed by the successive opening of alternate bays at each side for the best stilling action with minimum eddying. On a falling stage the closure will be in reverse order. When a gate bay is opened, all leaves are to be removed. The leaves will be handled by a gantry crane, and except for the top three central gate leaves, will be stored in slots at the top of the piers upstream from the service slots. Provision for storing the top three central gate leaves is made at the end of the structure. Except during the time necessary for removal of the leaves, discharge will not be passed over any of the lower gate leaves.

Overbank Structure

Upstream from the low-sill structure and sharing a common abutment of compacted fill will be the overbank structure, which is needed to provide the half of the required discharge capacity not furnished by the low-sill structure under the project flood conditions with the headwater elevation at 64.0 feet

and tailwater elevation at 62.5 feet. (Figure 5) A uniform weir crest elevation of 52.0 feet msl was selected as being representative of the general bank level in the vicinity. The ground surface at the site is about 4 feet lower than the elevation of the weir crest. Regulation of flow through this structure will be required to ensure that the combined discharge of the two structures will follow the adopted plan of operation. Hence provision of controls is necessary.

An inflow channel will be excavated for a distance of 106 feet riverward from the overbank structure and the remainder of the area between river and structure will be cleared to a width about 125 feet greater, both upstream and downstream, than the overall length of the spillway between training walls. The average distance from the river to the structure is 3,300 feet. Riprap pavement will be placed in the excavated inflow channel at the invert elevation of 45.0 feet for a distance of 40 feet upstream from the face of the structure, and extended around the wing walls and on the riverside slopes of the adjacent levee and joint abutment fill.

A stilling basin, designed to dissipate the energy of the design discharge of 6,780 cfs per bay, will extend downstream a distance of 65.5 feet measured from the heel of the weir. (Figure 6) It will contain two rows of baffle blocks 5 feet in height and will terminate with an end sill 4 feet in height. A shallow outflow channel with an invert elevation of 46.5 feet will be excavated from the end sill downstream for a distance of 150 feet. The upstream two-thirds of this channel will be protected with derrick stone and riprap paving.

The same bay width of 44 feet used with the lower structure has been retained for the overbank spillway. A total of 73 bays is to be provided. The piers will have a thickness of 2 feet and thus the gross length between training walls will be 3,356 feet. Control of flow is to be accomplished by hinged panels.

Existence of favorable foundation conditions with respect to shear and consolidation characteristics and of a relatively uniform nature made it feasible to plan this structure without a pile foundation. A relatively wide base of structure has been provided and the design incorporates both upstream and downstream cutoff keys. These features reduce bearing pressures to allowable values and insure against detrimental settlement and horizontal sliding.

Bridge piers at 46-foot centers divide the spillway structure into 73 bays. Expansion joints at the midpoints of alternate bays divide the structure into 35 typical monoliths 92 feet in length, and 2 end monoliths 75 feet in length.. Included in each monolith are two piers, weir, foundation slab, and the two cutoff keys. Two additional bays at each end of the spillway will be 36.5 feet center-to-center of piers and will be closed by reinforced concrete bulkheads to form permanent dams connecting the structure to the abutments. The stilling basin slab will be placed on a sand drainage blanket with perforated clay tile drain pipes, provided to prevent the formation of excessive uplift pressures by seepage water. Circulation of water and loss of foundation material will be prevented by rubber water stops placed in the expansion joints. The slab has been designed to have sufficient weight to resist flotation. As in the case of the low-sill structure, access across the structure will be provided by a two-land concrete bridge.

Each spillway bay will be equipped with fifteen of the hinged panels for control of discharge through the structure. The panels will be of treated timber and will be comprised of three members 11 1/2 inches wide, 10 inches

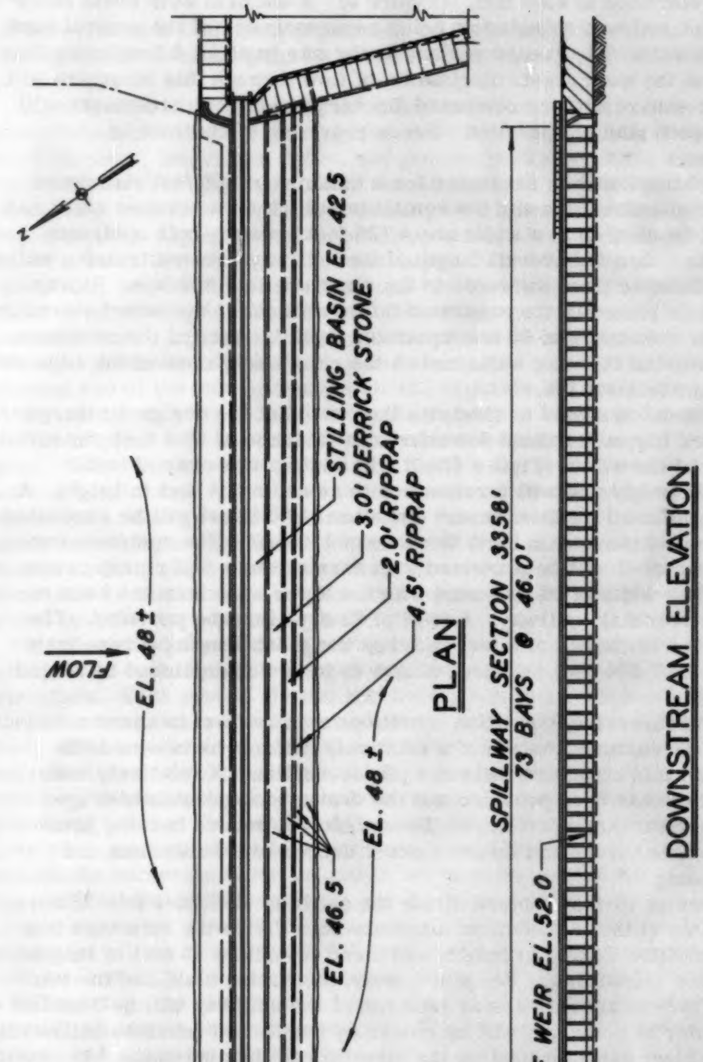

FIGURE 5 - OVERBANK CONTROL STRUCTURE

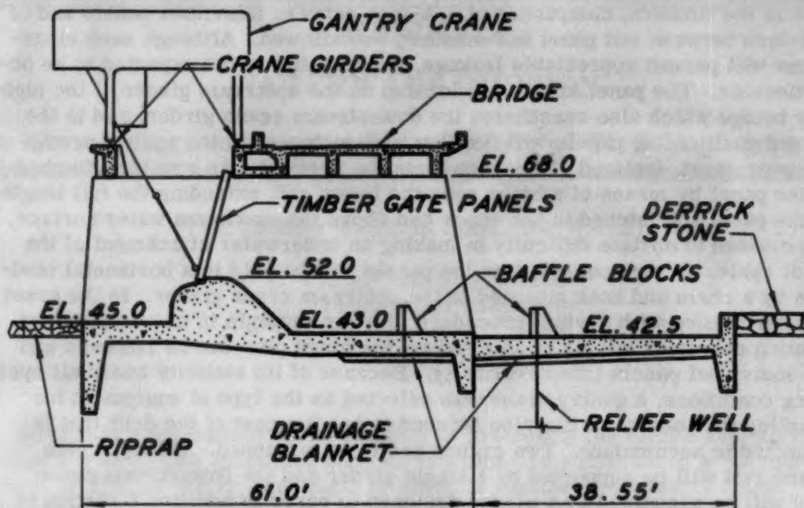

FIGURE 6 - SECTION OF OVERBANK CONTROL STRUCTURE

20 FT. 0 20 40 60 FT.

SCALE IN FEET

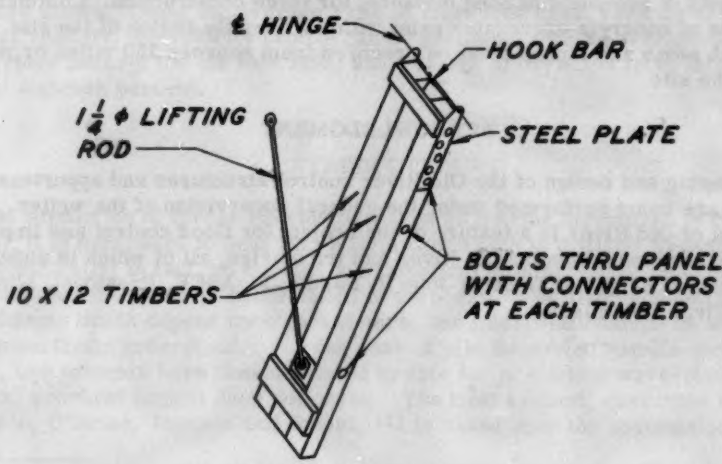

FIGURE 7 - TIMBER GATE PANEL

deep, and 18 feet long, fastened together. (Figure 7) To allow for inaccuracies in the timbers, clearances of 5/8-inch between individual panels and of 7/8-inch between end panel and masonry are allowed. Although such clearances will permit appreciable leakage, the quanitity is not expected to be objectionable. The panel hinges are located on the upstream girder of the highway bridge which also constitutes the downstream crane girder, and in the closed position the panels will bear on an 8-inch seat in the upstream edge of the weir crest, inclined 15 degrees from the vertical. An eye-bar attached to the panel by means of a hinge near the lower end, extending the full length of the panel and latched to the upper end above the maximum water surface, is provided to obviate difficulty in making an underwater attachment of the hoist cable. In the open position the panels will be held in a horizontal position by a chain and hook attached to the upstream crane girder. In the event of interference with normal procedure, due for example to accumulation of floating debris or failure of an eye-bar, the hinge pins can be removed and the individual panels lifted vertically. Because of its stability under all operating conditions, a gantry crane was selected as the type of equipment for handling the panels. It can also be used to handle most of the drift that is expected to accumulate. Two cranes are to be furnished. The upstream crane rail will be supported by a single girder and the downstream crane rail will be supported by a girder designed to carry in addition a portion of the two-land highway bridge and to take the reactions of the gate panels.

Materials for Construction

The required excavations for the control structures are expected to provide ample quantities of suitable materials for structure backfill and, within the limits of economic hauling distance, for levee construction. Commerical sources of concrete aggregates exist within a 50-mile radius of the site. Derrick stone and riprap must be procured from sources 250 miles or more from the site.

ACKNOWLEDGMENT

Planning and design of the Old River control structures and appurtenant works are being performed under the general supervision of the writer. Control of Old River is a feature of the project for flood control and improvement of the lower Mississippi River and tributaries, all of which is under the direction of Brigadier General John R. Hardin, M. ASCE, President, Mississippi River Commission.

JOURNAL

WATERWAYS DIVISION

Proceedings of the American Society of Civil Engineers

THE DESIGN WAVE IN SHALLOW WATER

R. L. Wiegel* and K. E. Beebe**
(Proc. Paper 910)

SYNOPSIS

Herein is a discussion of the problem of predicting the forces exerted by ocean waves on pile supported structures which are subjected to a "design wave" in shallow water, the height of which is limited by the water depth. This is the case for many locations in the Gulf of Mexico, the continental shelf of the Atlantic Seaboard, and offshore of Southern California. A summary of the solitary wave theory is presented, together with a comparison of predicted water particle velocities and laboratory measurements. A modified solitary wave theory is shown to predict the velocity profile of the water particles under the crest of a breaking wave with an average deviation of zero and a standard deviation of twenty-seven percent provided the actual crest velocities are known. Using the average ratio of measured crest velocity to theoretical velocity, the velocity profile can be predicted with an average deviation of thirteen percent (on the safe side) and a standard deviation from this average of eighteen percent.

INTRODUCTION

The prediction of forces exerted by waves on marine structures is complicated. It is necessary to deal with a fluid flow which is difficult to describe mathematically; in fact, no correct mathematical solutions have been obtained for the conditions usually encountered in the ocean. It is also necessary to use coefficients which depend upon parameters, the interrelationships of which are imperfectly understood. For the case of pile supported marine structures, two methods have been advanced to date for predicting wave-produced forces, provided impact does not occur. The first method, described by Morison, O'Brien, Johnson and Schaaf,(11) is based upon the assumption that

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the force consists of two parts (a drag force due to the water particle velocity and an inertia force due to water particle acceleration) and that the solution can be obtained by treating each component separately and adding the solutions linearly to obtain the total force. The equation contains two coefficients which have to be determined empirically. The second method, described by Crooke,[2] is based upon the study by Iversen and Balent[7] of the forces exerted upon a body in accelerated motion. It is assumed that there is a linear dependence of velocity upon acceleration so that the force can be expressed as the product of one coefficient, the fluid density, the projected pile area, and the square of the particle velocity. The coefficient is related primarily to Iversen's Modulus (the product of the particle acceleration and the pile diameter divided by the square of the water particle velocity) and Reynolds Number. Published data indicate that improvements in both methods are necessary. A review of work in similar fields and of the limited data on wave forces shows that it will be necessary to determine the relationships among the coefficients in the force equations and other factors such as the degree of "upstream" turbulence,[4] pile roughness,[4] and past history of the flow (indicated by the differences between the data obtained by Iversen and Balent[7] and by Luneau.[9])

Both methods of force prediction depend upon a knowledge of water particle motion[19,20] and empirically determined coefficients. The coefficients are combined in the force formulae with theoretical equations for water particle velocity and acceleration. The fact that the values of the coefficients have to be determined by the same means (that is, relying upon theoretical values of fluid motion rather than measured values) leads to the difficulty that if the equations do not correctly predict the particle motion, then the coefficients can be used only under similar conditions. Theoretically the equations are valid only for limited conditions, for waves of finite height in deep water, and for low amplitude waves (compared with both wave length and water depth) in shallow water.[10,14,19] Few data are available with which to determine the degree of accuracy of these equations even under the most ideal conditions.[3,12,16]

Waves encountered in the ocean present a more difficult problem than the uniform waves dealt with both theoretically and in most laboratory tests. Because they are non-uniform both in the direction of motion and along the crests they should be treated by means of a linear (or non-linear, probably, for the design wave) superposition of component waves rather than by the concept of a representative train of waves with one height and length. There appears to be no published data from which to determine the errors introduced by using the representative wave approach in computing the wave forces, although Fuchs[5] has studied nonuniform waves to a limited extent for a similar application. Further, the so-called "design wave" will almost always be under the action of wind, so there will be a considerable wind-induced current near the surface.[16] Reid and Bretschneider[14] have developed a formula which can be used to predict the force on a pile where a current is present. However, at present the lack of quantitative knowledge of these surface currents precludes the use of this formula.

In addition to problems of this type, there is the one encountered in certain regions (such as the Gulf of Mexico) where the design wave, as predicted by meteorologists for, say, a twenty- or fifty-year life of a structure, is of such a large magnitude compared with the water depth that depth is the

limiting factor. There is no theory developed which correctly describes the characteristics of such a wave. Instead, it is necessary to use an approximation and to define its limitations, so that the design engineer can properly choose a factor of safety.

Solitary Wave Theory

The problem of determining the forces exerted on piles by waves which are breaking, or nearly breaking, due to depth limitations, is extremely difficult to handle. There is no satisfactory theory which expresses the water particle velocities and accelerations associated with the motion of periodic waves in the region in which the wave breaks. Munk[13] has suggested that the solitary wave theory might be used as a satisfactory approximation; he states that ocean waves have certain of the characteristics of a solitary wave; for example, the wave consists of narrow crests separated by long flat troughs, and the velocity depends almost entirely upon the water depth and wave height. Ocean waves are, however, periodic to the extent that one follows another with an interval which can be described roughly as a "period," and furthermore they have troughs which lie below the still water level. However, the portion of the wave below the still water level in shallow water is much less than is the case in deep water.[18] Further, Keller[8] has shown that the solitary wave theory represents an extreme case for certain types of periodic waves.

As the solitary wave theory, originally developed by Boussinesq and extended and modified by McCowan and other later investigators, has been well summarized by Munk,[13] it will not be presented in detail in this paper; rather, the reader is referred to Munk's paper.

The equation for the horizontal component of particle velocity, u, is given by:

$$u = CN \frac{1 + \cos MZ \cosh MX}{(\cos MZ + \cosh MX)^2} \tag{1}$$

where M and N are defined as functions of γ by the equations:

$$\gamma = \frac{N}{M} \tan^{\frac{1}{2}} \left[M (1 + \gamma) \right] \tag{2a}$$

$$N = \frac{2}{3} \sin^2 \left[M \left(1 + \frac{2}{3}\gamma\right) \right] \tag{2b}$$

where:

γ = H/h = wave height/water depth below the trough
C = wave crest velocity
Z = z/h = vertical distance above the bottom/water depth below the trough
X = x/h = horizontal distance from the crest/water depth below the trough.

For the case of the solitary wave of maximum steepness a different expression is needed to describe the velocity in the vicinity of the crest. This is:[14]

$$\frac{u}{C} = 1 - 0.8\,(1.78 - Z)^{\frac{1}{2}} - 0.067\,(1.78 - Z)^{3/2}\ . \tag{3}$$

It is apparent that the solution of the set of equations 1, 2a and 2b must be obtained by successive approximations. Fortunately the solution has been presented in graphical form by Munk[13] (see Figure 1) and has been tabulated for the maximum value of γ ($\gamma_{max} = 0.78$) by Reid and Bretschneider.[16] This table has been reproduced in Table I. The values presented were computed by means of Equation 3 where necessary.

Reid and Bretschneider[14] gave the equation for the horizontal component of water particle acceleration, du/ dt, as:

$$\frac{du}{dt} = g\left(1+\frac{H}{h}\right) NM (\sinh MX) \left\{ \frac{2 - N + (\cos MZ)\ \left[(\cosh MX) - (\cos MZ)\right]}{\left[(\cos MZ) + (\cosh MX)\right]^3} \right\} \quad (4)$$

where g is the acceleration of gravity. Reid and Bretschneider also tabulated values of acceleration for the maximum value of γ ($\gamma_{max} = 0.78$). These values have been presented in Table II.

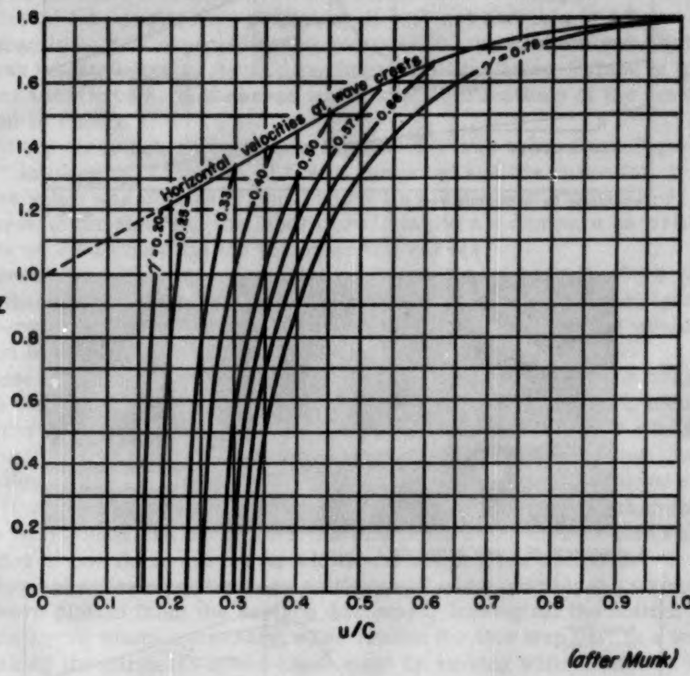
Comparison with Laboratory Data

Before Equations 1, 3 and 4 can be used in the wave force formula, it is necessary to determine the accuracy with which the solitary wave theory predicts particle motion. The data presented herein were obtained during an extensive laboratory study of the kinematics of breaking waves[6,12] which was performed at the University of California, Berkeley. The data were obtained by inserting non-miscible drops of fluid with the same specific gravity as water in the wave channel and taking motion pictures of their motion when waves were generated. Details of the laboratory procedure (see Figures 2 and 3 for the notations and the type of data obtained) and the general results have been presented elsewhere[6,12] and will not be repeated. Only the data basic to the problem of predicting wave forces are given, together with the heretofore unpublished detailed data on particle velocities (Table III and Figures 4-9).

The controlling parameter in the solitary wave theory is the ratio of wave height to the undisturbed water depth, γ. The maximum value this parameter can have is 0.78 as the wave becomes unstable for this value. Waves encountered in the ocean are not solitary waves and a portion of the waves, the troughs, lie below the still water level. Munk[13] suggested that many characteristics of breaking waves in shallow water could be described by the solitary wave theory provided γ were taken as the ratio of the wave height to the depth below the trough. As can be seen in Table III (Column 11) this value was always greater than 0.78 for the conditions tested, although there was a trend to lower values for flatter beaches. A detailed examination of the data indicated that a better criterion would be the ratio of the portion of the wave above the still water level to the still water depth (see Table III, Column 12), γ^*.

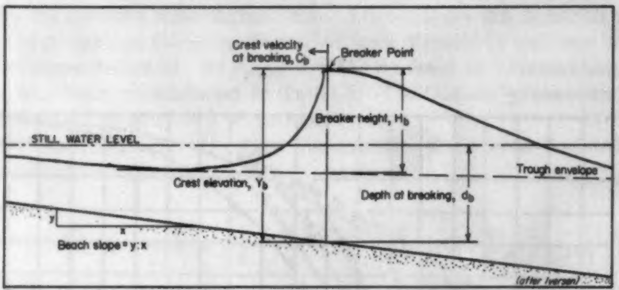
In Figures 4-6 are presented the measured values of u/C, where u is the horizontal component of particle velocity and C is the wave crest velocity as obtained by use of the equation:

$$C = \sqrt{gY_b} \quad (5)$$



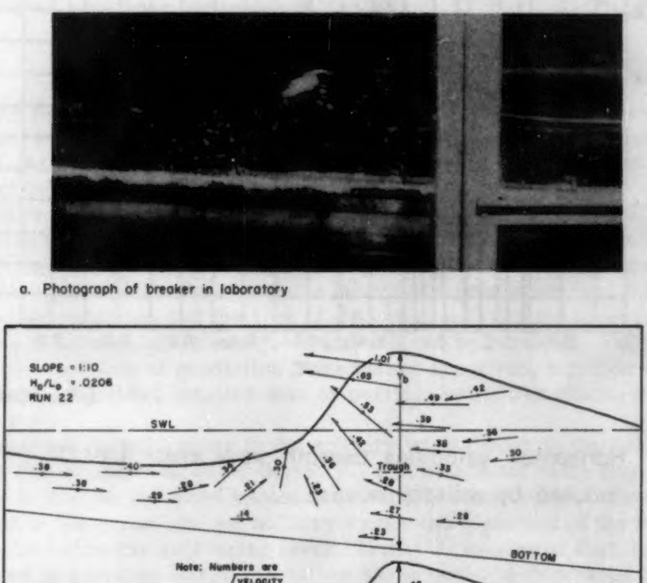
Horizontal velocities beneath wave crest (X = O) induced by solitary waves.

FIGURE I



Terminology for a Breaking Wave

FIGURE 2



a. Photograph of breaker in laboratory

b. Kinematic data

Kinematics of a Breaking Wave

FIGURE 3

where Y_b is the elevation of the breaker crest above the bottom. Data are presented for beach slopes of 1:10, 1:20 and 1:50. An attempt was made to obtain the values of the horizontal component of water particle velocity along a vertical line directly beneath the crest of the wave just as it was breaking. Because of the difficulty in determining this instant exactly, and because of the scatter of the nonmiscible particles, it was not possible to adhere closely to this practice. It is expected that a good deal of the scatter in the data presented can be attributed to these experimental difficulties. Values of u/C are compared with three sets of curves which are modifications of the curves presented in Figure 1.

One of the three sets of curves in Figures 4-6 was taken from Figure 1 using γ^* in place of γ; that is, γ^* was computed and the curve for γ of this same value was obtained from Figure 1 and drawn in Figures 4-6. As can be seen in the figures, the laboratory data did not compare as well with the curve of γ^* as they did for the other two curves.

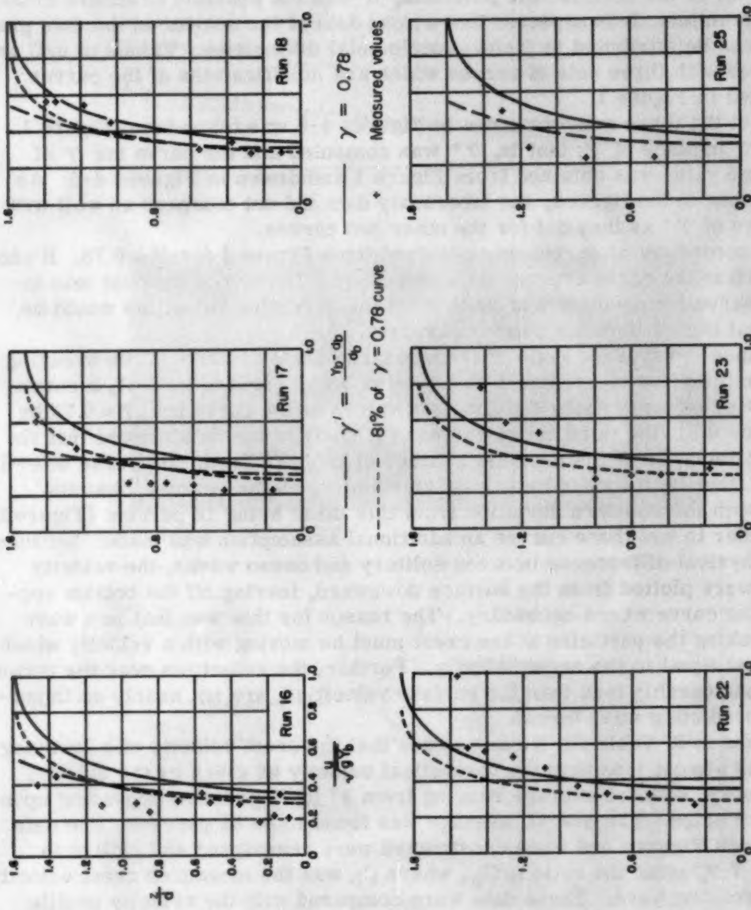
The second set of curves was obtained from Figure 1 for $\gamma = 0.78$. It can be seen that the curve represents a maximum. The use of it would lead to the conservative prediction of velocities; the predicted velocities would be somewhat higher than the measured values.

Because the average ratio of the measured crest velocity of the breaking waves to the velocity predicted by Equation 5 was found to be 0.81, a curve was plotted by multiplying the abscissa values of the curve for $\gamma = 0.78$ by the factor 0.81 (the third set of curves). A study of the data showed that the average deviation of the measured values of $u/\sqrt{gY_b}$ from this curve was -13 percent (that is, the velocity values were lower, on the average, than the curve) with the standard deviation from this mean being 18 percent (Figure 10).

In order to use these curves an additional assumption was made: because of the physical differences between solitary and ocean waves, the velocity curves were plotted from the surface downward, leaving off the bottom section of the curve where necessary. The reason for this was that in a wave just breaking the particles at the crest must be moving with a velocity which is at least equal to the crest velocity. Further, the velocities near the bottom, being considerably less than the surface velocities, are not nearly so important in predicting wave forces.

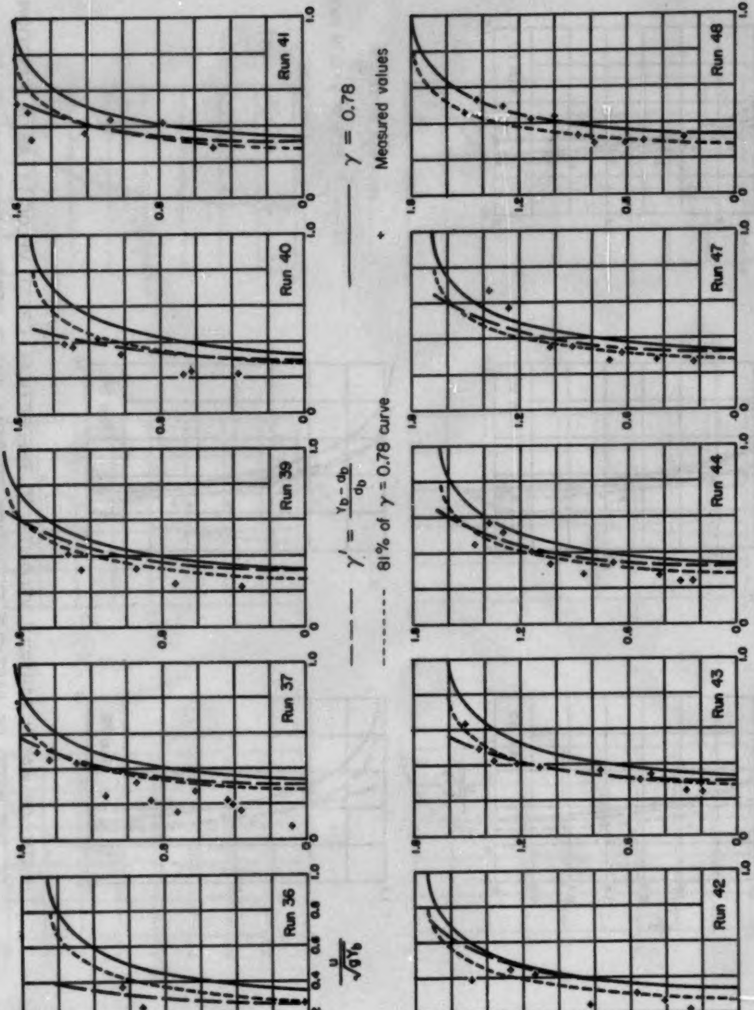
In Column 9, Table III, it can be seen that the crest velocity of a breaking wave was almost less than the theoretical velocity as given by the solitary wave theory, with the average ranging from 87 to 79 percent, depending upon the beach slope. The overall average was found to be 81 percent. The data from which Figures 4-6 were constructed were reanalyzed and plotted in Figures 7-9, using the ratio u/C_b, where C_b was the measured crest velocity of the breaking wave. These data were compared with the velocity profile curve predicted by the solitary wave theory for the maximum value of $\gamma = 0.78$. The agreement appears to be satisfactory with the average deviation between the measured values of u/C_b and the theoretical curve for $u/\sqrt{gY_b}$ being zero, and the standard deviation being 27 percent (see Figure 11). The curve was "forced fit," as described previously by plotting from the surface downward.

It appears from the data presented herein that the solitary wave theory can be used to predict the horizontal component of water particle velocity with a fair degree of accuracy providing the necessary modifications are made.



RATIO OF WATER PARTICLE VELOCITY TO THEORETICAL CREST VELOCITY AS A FUNCTION OF THE RATIO OF PARTICLE ELEVATION TO STILL WATER DEPTH - Beach slope of 1:20

FIGURE 4



RATIO OF PARTICLE VELOCITY TO THEORETICAL CREST VELOCITY AS A FUNCTION OF THE RATIO OF PARTICLE ELEVATION TO STILL WATER DEPTH - Beach slope of 1:20

HYD-7184

FIGURE 5

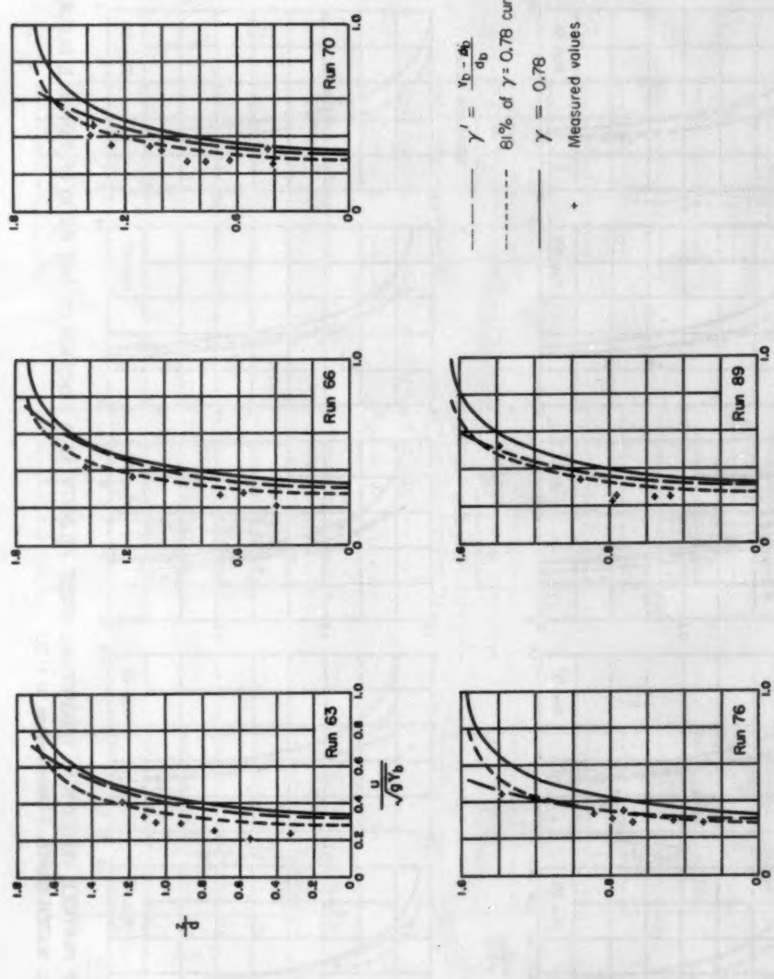
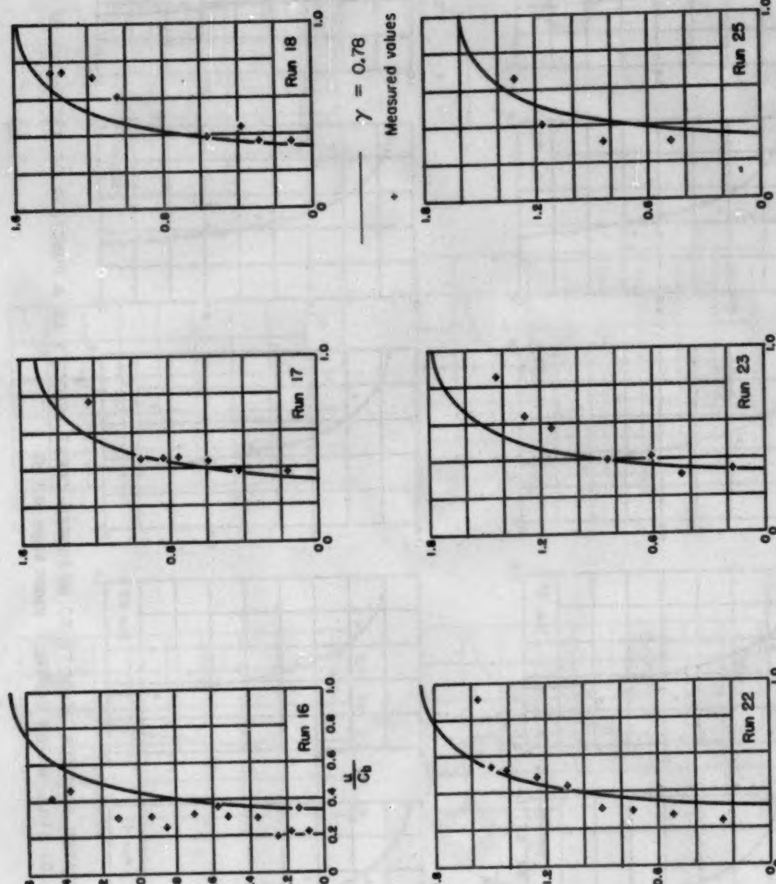


FIGURE 6

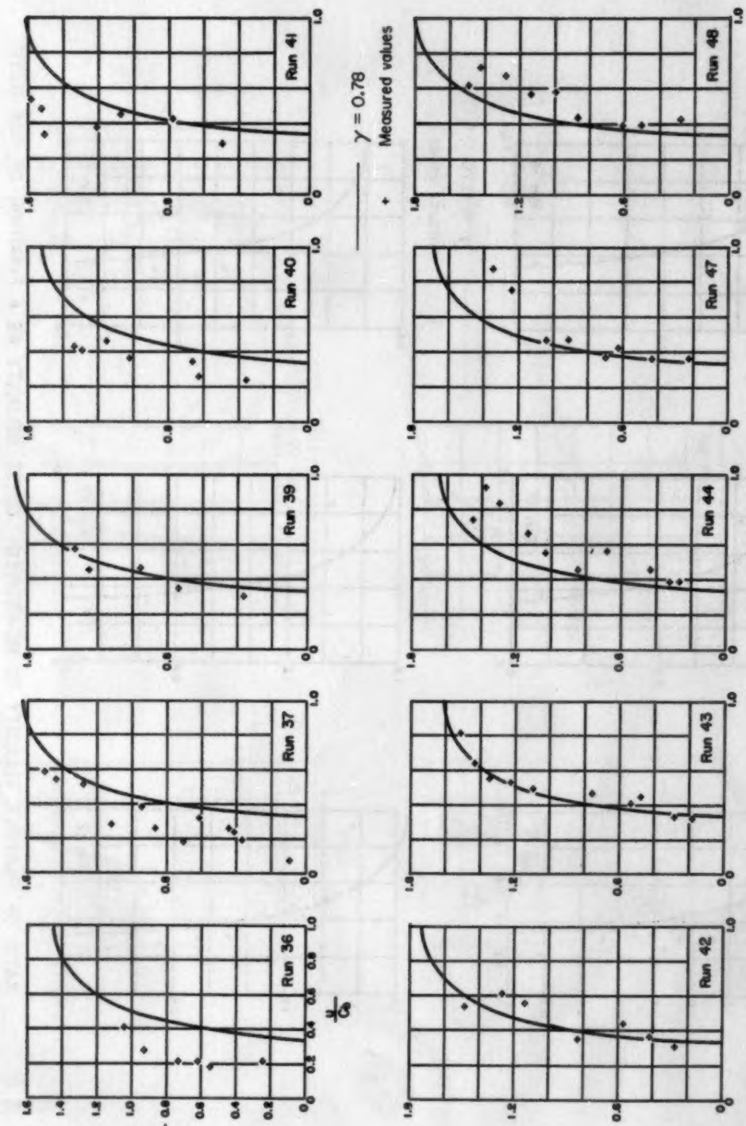
RATIO OF WATER PARTICLE VELOCITY TO THEORETICAL CREST VELOCITY AS A FUNCTION OF THE RATIO OF PARTICLE ELEVATION TO STILL WATER DEPTH - Beach slope of 1:50



RATIO OF PARTICLE VELOCITY TO MEASURED CREST VELOCITY AS A FUNCTION OF THE RATIO OF PARTICLE ELEVATION TO STILL WATER DEPTH - Beach slope of 1:10

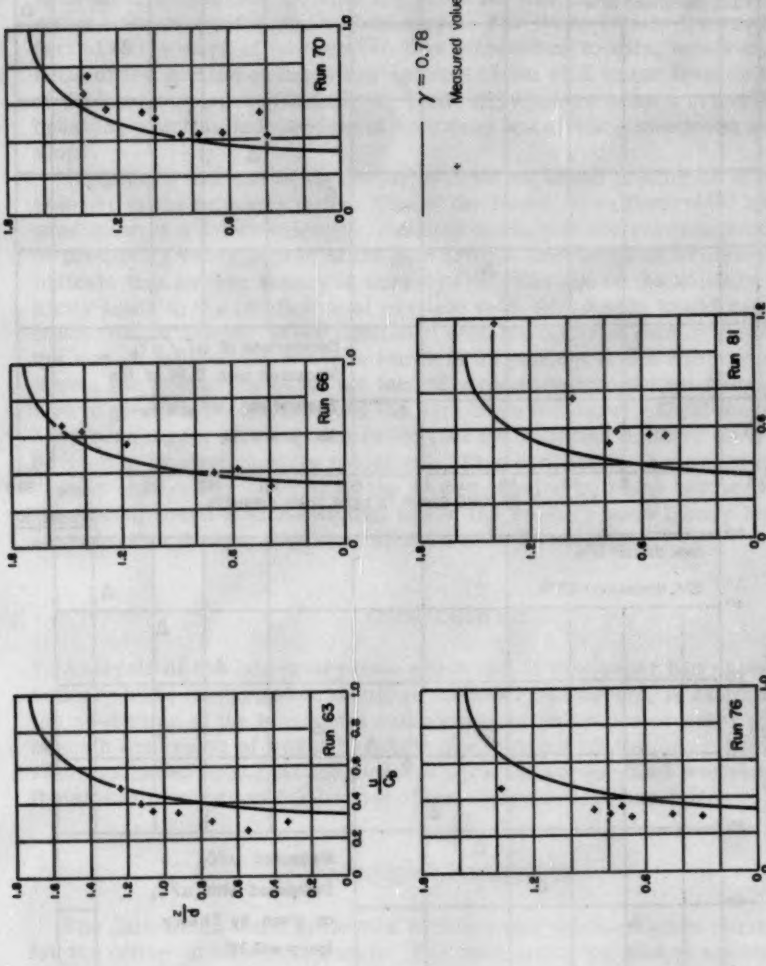
FIGURE 7

HYD-7166



RATIO OF WATER PARTICLE VELOCITY TO MEASURED CREST VELOCITY AS A FUNCTION OF THE RATIO OF PARTICLE ELEVATION TO STILL WATER DEPTH - Beach slope of 1:20

FIGURE 8



RATIO OF WATER PARTICLE VELOCITY TO MEASURED CREST VELOCITY AS A FUNCTION OF THE RATIO OF PARTICLE ELEVATION TO STILL WATER DEPTH - Beach slope of 1:50

HYD-7188

FIGURE 9

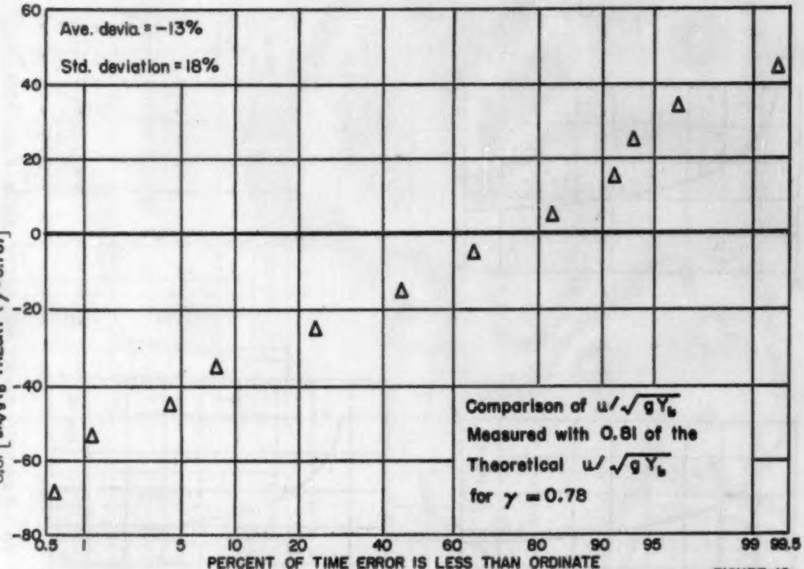
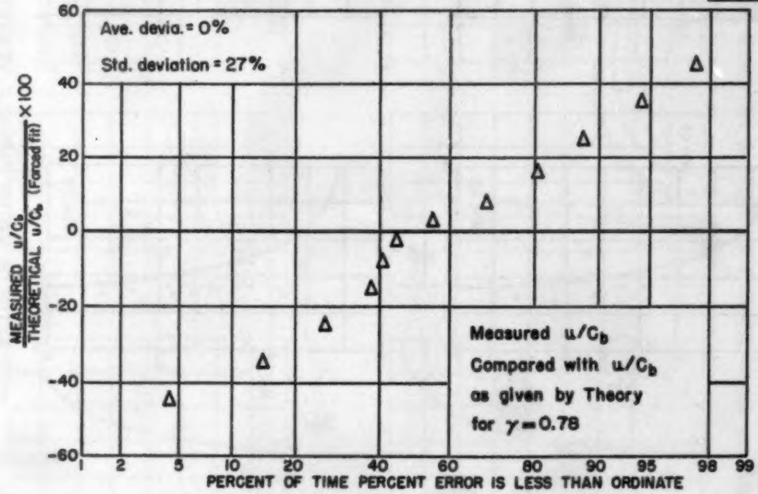


FIGURE 10



HYD-7189

FIGURE 11

In order to apply the solitary wave theory to the physical dimensions of the design waves predicted by meteorologists, considerable knowledge is needed of certain characteristics, such as the ratio of limiting depth of water to breaker height (see Column 10, Table III) and its dependence upon the deep water wave steepness and bottom slope. For these details the reader is referred to the work of Iversen.[7] It is interesting to note, however, that the ratio of the portion of the breaking wave above still water level to the height of the breaking wave (Column 13, Table III) appears to be a criterion for breaking which is independent of both deep water wave steepness and beach slope.

It appears that one of the major difficulties is the prediction of wave crest velocity of the breaking wave. Use of the linear wave theory[20] leads to the prediction of a lower velocity. Detailed studies of the reverse problem, that of predicting water depths in the surf from a knowledge of breaker velocities, indicate that neither theory is correct.[15] The use of the solitary wave velocity leads to the prediction of particle velocities which would result in a conservative design. When combined with the breaker data of Iversen to give the new particle velocity curve which is 81 percent of the solitary wave theory curve, the prediction would not lead to a conservative design when consideration is given to the deviation of the data from the curve. Until more studies have been made, it is recommended that the modified solitary wave theory be used to predict particle velocities. This conclusion is strengthened by the results obtained by Carr[1] on the forces exerted on plane barriers by breaking waves, which also shows that use of the solitary wave theory leads to a conservative design, providing shock forces do not occur.

CONCLUSIONS

Analysis of the laboratory data presented in this paper has shown that the solitary wave theory, when modified as described herein, is satisfactory for the prediction of the horizontal components of velocities of water particles beneath the crests of waves breaking due to depth limitations. This conclusion was based upon data for uniform periodic waves which were not under the stress of wind and for bottom slopes between 1:10 and 1:50.

ACKNOWLEDGMENTS

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REFERENCES

1. Carr, John H. Breaking wave forces on plane barriers, Calif. Inst. of Tech. Hydrodynamics Lab., Pasadena, Calif. Report No. E-11.3, 25 pp, 29 plates, Nov. 1954, (Unpublished).

2. Crooke, R. C., Re-analysis of existing wave force data on model piles, TM-71 Beach Erosion Board, Corps of Engineers, U.S. Army, 19 pp, April 1955.

3. Elliott, John G., Interim report, Calif. Inst. of Tech., Contracts NOy-12561, July 1953.

4. Fage, A. and Warsap, J. H., The effects of turbulence and surface roughness on the drag of a circular cylinder, Aeronautical Res. Comm., Air Ministry, Great Britain, R & M, No. 1283, 14 pp, Oct. 1929.

5. Fuchs, R. A., On the theory of irregular waves, Proc. First Conf. on Ships and Waves, Council on Wave Res. and Soc. of Naval Architects and Marine Engrs., pp 1-11, 1955.

6. Iversen, H. W., Laboratory study of breakers, Gravity Waves, National Bu. Standards, Cir. No. 521, pp 9-32, Nov. 1952.

7. Iversen, H. W., and Balent R., A correlating modulus for fluid resistance in accelerated flow, Jour of Applied Physics, Vol. 33, no. 3, pp 324-328, March 1951.

8. Keller, Joseph B., The solitary wave and periodic waves in shallow water. Ocean Surface Waves, Annals of the N.Y. Acad. of Sci., Vol. 51, art. 3, pp 345-350, May 1949.

9. Luneau, Jean, Sur l'effect d'inertie des sillages de corps se déplacant dans un fluide d'un mouvement uniformémente accéléré, Academié des Sci. Paris, Compte Rendus, Vol. 227, no. 17, pp 823-824, 27 Oct. 1948.

10. Miche, M. Undulatory movements of the sea in constant or decreasing depths. A translation from French, Univ. of Calif., Berkeley, IER Tech. Report Ser. 3, Issue 363, 96 pp June 1954, (unpublished).

11. Morison, J. R., O'Brien, M. P., Johnson, J. W., and Schaaf, S. A., The force exerted by surface waves on piles. Pet. Trans. AIME, Vol. 189, TP 2846, pp 149-154, 1950.

12. Morison, J. R. and Crooke, R. C., The mechanics of deep water, shallow water and breaking waves, Beach Erosion Board, Corps of Engrs. U.S. Army Tech. Memo., 40, 14 pp, March 1953.

13. Munk, Walter H., The solitary wave theory and its application to surf problems, Ocean Surface Waves, Annals of the N.Y. Acad. of Sci., Vol. 51, art. 3, pp 376-423, May 1949.

14. Reid, R. O. and Bretschneider, C. L., The design wave in deep water or shallow water, storm tide, and forces on vertical piling and large submerged objects, A & M. College of Texas, Dept of Oceanography, Tech. Report on Contract NOy-27474, DA-49-005-eng-18, and N7onr-48704, 36 pp, appendices and graphs, Feb. 1954, (unpublished).

15. Sibul, O., Laboratory study of depth determination: effect of offshore bars, University of Calif., Berkeley, IER Series 74, Issue 8, 13 pp, 15 plates, Sept. 1953, (unpublished).

16. Suquet, F. and Wallet, A., Basic experimental wave research, Proc. Minn. International Hydraulics Convention, IAHR and ASCE, pp 173-191, Aug. 1953.

17. Van Dorn, W. G., Wind stress over water, Univ. of Calif., Scripps Inst. of Oceanography, La Jolla, Calif. Ref. No. 52-60, 23 pp, Nov. 1952, (unpublished).

18. Wiegel, R. L., Experimental study of surface waves in shoaling water, Trans. Amer. Geophys. Union, Vol. 31, No. 3, pp 377-385, June 1950.

19. Wiegel, R. L., Gravity waves: tables of functions, Council on Wave Research, Engineering Foundation, Berkeley, Calif., 30 pp, Feb. 1954.

20. Wiegel, R. L. and Johnson, J. W., Elements of wave theory, Proc. First Conf. on Coastal Engineering, Council on Wave Research, the Engineering Foundation, Berkeley, pp 5-21, 1951.

TABLE I

SUMMARY OF VELOCITIES IN THE HIGHEST SOLITARY WAVE (H/h = 0.78)
Velocities expressed in terms of $(u/C)^2$

x/h	0	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.2	2.6	3.0	3.4	5.0
z'/h															
1.78	1.000														
1.70	.599														
1.60	.436	.373													
1.50	.336	.302	.255												
1.40	.281	.260	.229	.185	.137										
1.30	.245	.230	.205	.168	.127	.090									
1.20	.216	.206	.185	.154	.119	.087	.063	.042	.029						
1.10	.194	.186	.169	.143	.112	.084	.063	.044	.031	.021	.009				
1.00	.176	.170	.156	.133	.106	.082	.063	.046	.032	.023	.011	.005	.002	.001	.0001
.90	.161	.156	.144	.124	.101	.080	.062	.047	.033	.024	.012	.006	.003	.001	.0001
.80	.148	.144	.134	.116	.096	.078	.062	.048	.034	.025	.013	.007	.003	.001	.0001
.70	.138	.134	.126	.110	.093	.077	.061	.048	.035	.026	.014	.007	.004	.002	.0001
.60	.130	.127	.119	.106	.089	.075	.061	.048	.036	.027	.014	.008	.004	.002	.0001
.50	.124	.121	.114	.101	.086	.074	.060	.048	.037	.028	.015	.008	.004	.002	.0001
.40	.119	.116	.109	.098	.084	.073	.060	.048	.037	.028	.016	.009	.004	.002	.0001
.30	.115	.112	.105	.096	.082	.072	.059	.048	.037	.028	.017	.009	.004	.002	.0001
.20	.112	.109	.102	.094	.081	.072	.059	.048	.038	.029	.017	.009	.004	.002	.0001
.10	.110	.107	.100	.093	.079	.071	.059	.048	.038	.029	.018	.009	.004	.002	.0001
0	.109	.106	.099	.092	.078	.070	.058	.048	.038	.029	.018	.009	.004	.002	.0001
$(z'/h)_{surf.}$	1.78	1.67	1.57	1.48	1.41	1.35	1.29	1.25	1.21	1.18	1.13	1.08	1.05	1.03	1.007
$(u/C)^2_{surf.}$	1.00	.430	.276	.201	.138	.092	.062	.041	.029	.020	.009	.004	.002	.001	.0001

After Reid and Bretschneider, McCowan Theory
Revised May 1955.

TABLE II

SUMMARY OF ACCELERATIONS IN THE HIGHEST SOLITARY WAVE ($H/h = 0.78$)
Accelerations expressed as fractions of g, i.e. $\frac{1}{g}\frac{du}{dt}$

x/h	0	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.2	2.6	3.0	3.4	5.0
z'/h															
1.78	0														
1.70	0														
1.60	0	.212													
1.50	0	.173	.310												
1.40	0	.142	.259	.335	.352										
1.30	0	.118	.218	.285	.310	.307									
1.20	0	.100	.185	.247	.273	.277	.262	.236	.208						
1.10	0	.086	.159	.213	.240	.249	.243	.223	.200	.173	.124				
1.00	0	.073	.137	.184	.214	.225	.225	.209	.192	.171	.128	.091	.067	.049	.012
.90	0	.062	.119	.162	.191	.205	.209	.198	.185	.168	.130	.094	.070	.051	.013
.80	0	.054	.104	.144	.171	.187	.193	.187	.178	.164	.131	.097	.073	.053	.014
.70	0	.048	.093	.129	.155	.172	.180	.178	.171	.160	.131	.099	.075	.054	.015
.60	0	.043	.083	.117	.142	.159	.168	.170	.165	.157	.131	.101	.078	.056	.015
.50	0	.039	.076	.108	.131	.149	.159	.163	.160	.154	.131	.103	.080	.057	.016
.40	0	.036	.070	.100	.123	.141	.152	.157	.157	.151	.131	.104	.082	.058	.016
.30	0	.034	.065	.094	.117	.135	.147	.152	.154	.149	.131	.106	.083	.059	.016
.20	0	.032	.062	.090	.113	.131	.143	.149	.151	.147	.130	.107	.084	.060	.017
.10	0	.031	.060	.088	.111	.128	.141	.147	.149	.146	.130	.108	.084	.061	.017
0	0	.031	.060	.087	.110	.127	.140	.146	.148	.145	.130	.109	.084	.062	.017
$(z'/h)_{surf.}$	1.78	1.67	1.57	1.48	1.41	1.35	1.29	1.25	1.21	1.18	1.13	1.08	1.05	1.03	1.007
$(\frac{1}{g}\frac{du}{dt})_{surf.}$	0	.242	.347	.380	.357	.321	.280	.243	.209	.174	.122	.088	.065	.049	.012

After Reid and Bretschneider, McCowan Theory
Revised May 1955.

Table III A

LABORATORY WAVE DATA: BEACH SLOPE 1:10.

Run	Wave Per. T sec	Wave Ht. H_m^* ft.	Still water depth d_m^* ft	Ht. break Wave H_b ft	Crest elev. at break. Y_b ft	Water depth break d_b ft.	Crest veloc at break C_b ft/sec	$\frac{C_b}{\sqrt{gY_b}}$	$\frac{d_b}{H_b}$	$\frac{H_b}{Y_b-H_b}$	$\frac{Y_b-d_b}{d_b}$	$\frac{Y_b-d_b}{H_b}$
4	1.00	0.391	2.30	0.40	0.75	0.41	-	-	1.03	1.14	0.83	0.85
5	1.00	0.391	2.30	0.40	0.75	0.41	-	-	1.03	1.14	0.83	0.85
22	1.51	0.220	2.23	0.37	0.56	0.30	4.65	1.09	0.81	1.95	0.87	0.70
15	1.73	0.231	2.25	0.36	0.60	0.32	-	-	0.89	1.50	0.88	0.78
25	1.00	0.400	2.33	0.35	0.72	0.45	3.55	0.74	1.28	0.95	0.60	0.77
13	0.92	0.250	2.23	0.26	0.52	0.33	3.10	0.76	1.27	1.00	0.58	0.73
2	1.98	0.140	2.24	0.31	0.51	0.30	-	-	0.97	1.55	0.70	0.68
23	1.98	0.131	2.23	0.23	0.46	0.26	3.55	0.92	0.90	1.71	0.77	0.69
17	0.80	0.200	2.23	0.21	0.44	0.29	2.74	0.73	1.38	0.91	0.52	0.71
16	1.11	0.168	2.23	0.22	0.39	0.22	3.40	0.96	1.00	1.29	0.68	0.77
10	1.27	0.129	2.17	0.22	0.38	0.18	-	-	0.82	1.38	1.11	0.91
7	1.26	0.114	2.17	0.19	0.32	0.16	-	-	0.84	1.46	1.00	0.84
8	1.45	0.123	2.17	0.20	0.32	0.18	-	-	0.90	1.67	0.78	0.70
27	1.26	0.085	2.15	0.16	0.27	0.14	-	-	0.88	1.45	0.93	0.81
20	2.10	0.113	2.22	0.23	0.47	0.28	-	-	1.22	0.96	0.68	0.83
24	2.50	0.111	2.22	0.24	0.38	0.24	3.45	0.88	1.00	1.71	0.56	0.58
					Average		=	0.87	1.01	1.36	0.78	0.76

Table III B

LABORATORY WAVE DATA; BEACH SLOPE 1:20

Run	Wave Per. T sec	Wave Ht. H_m^* ft.	Still water depth d_m^* ft	Ht. break Wave H_b ft	Crest elev. at break. Y_b ft	Water depth break d_b ft.	Crest veloc at break C_b ft/sec	$\frac{C_b}{\sqrt{gY_b}}$	$\frac{d_b}{H_b}$	$\frac{H_b}{Y_b-H_b}$	$\frac{Y_b-d_b}{d_b}$	$\frac{Y_b-d_b}{H_b}$
31	1.40	0.336	1.80	0.42	0.86	0.53	-	-	1.26	0.95	0.62	0.79
34	1.50	0.298	1.60	0.40	0.76	0.46	-	-	1.15	1.11	0.65	0.75
30	1.59	0.256	1.60	0.40	0.77	0.48	-	-	1.20	1.08	0.60	0.73
29	1.89	0.225	1.57	0.38	0.72	0.44	-	-	1.16	1.12	0.64	0.74
28	2.24	0.193	1.57	0.36	0.65	0.39	-	-	1.08	1.24	0.67	0.72
46	1.04	0.333	1.75	0.35	0.79	0.54	4.35	0.86	1.54	0.80	0.46	0.71
45	1.15	0.305	1.60	0.31	0.61	0.39	-	-	1.26	1.03	0.56	0.71
42	1.26	0.260	1.57	0.33	0.59	0.34	3.13	0.72	1.03	1.27	0.74	0.76
44	1.33	0.238	1.60	0.30	0.56	0.34	2.60	0.61	1.17	1.15	0.65	0.73
43	1.41	0.202	1.56	0.27	0.53	0.33	3.20	0.77	1.22	1.04	0.61	0.74
47	1.67	0.178	1.51	0.27	0.49	0.29	3.00	0.76	1.07	1.23	0.69	0.74
48	1.93	0.144	1.49	0.25	0.45	0.25	2.90	0.76	1.00	1.25	0.80	0.80
36	0.74	0.214	1.55	0.19	0.42	0.29	3.40	0.92	1.53	0.83	0.45	0.68
37	0.93	0.206	1.50	0.21	0.44	0.27	3.17	0.84	1.29	0.91	0.63	0.81
40	1.03	0.185	1.50	0.18	0.38	0.25	3.20	0.91	1.39	0.90	0.52	0.72
39	1.12	0.165	1.50	0.19	0.38	0.23	2.50	0.71	1.21	1.00	0.65	0.79
38	1.17	0.145	1.50	0.20	0.36	0.21	-	-	1.06	1.25	0.71	0.75
35	1.34	0.110	1.50	0.14	0.27	0.16	-	-	1.14	1.08	0.69	0.79
41	1.55	0.094	1.47	0.15	0.29	0.18	3.03	0.99	1.20	1.07	0.61	0.73
					Average		=	0.81	1.21	1.07	0.63	0.75

* Measured in the constant depth portion of the wave channel.

TABLE III C

LABORATORY WAVE DATA; BEACH SLOPE 1:50

Run	T sec.	H_m* ft.	d_m* ft	H_b ft	Y_b ft	d_b ft	C_b ft/sec	$\frac{C_b}{\sqrt{gY_b}}$	$\frac{d_b}{H_b}$	$\frac{H_b}{Y_b-H_b}$	$\frac{Y_b-d_b}{d_b}$	$\frac{Y_b-d_b}{H_b}$
78	2.45	0.232	1.54	0.353	-	-	-	-	-	-	-	-
81	2.65	0.243	1.54	0.398	0.840	0.513	2.30	0.44	1.28	0.91	0.65	0.83
58	1.00	0.348	1.54	0.303	0.637	0.403	3.00	0.67	1.33	0.91	0.58	0.77
70	1.13	0.282	1.54	0.287	0.585	0.350	2.80	0.64	1.17	1.05	0.69	0.80
63	1.17	0.243	1.54	0.274	0.554	0.321	3.50	0.83	1.19	0.96	0.72	0.85
68	1.62	0.228	1.54	0.268	0.522	0.306	3.50	0.86	1.15	1.08	0.68	0.78
66	1.74	0.190	1.54	0.283	0.565	0.334	2.85	0.60	1.18	0.97	0.73	0.86
82	2.65	0.186	1.54	0.320	0.691	0.422	-	-	1.31	0.86	0.64	0.84
62	0.81	0.301	1.54	0.250	0.555	-	-	-	-	-	-	-
61	0.90	0.279	1.54	0.222	0.497	0.326	3.90	0.97	1.50	0.79	0.52	0.77
74	0.95	0.222	1.54	0.191	0.394	0.228	2.75	0.78	1.53	0.95	0.34	0.53
59	1.00	0.230	1.54	0.218	0.455	-	-	-	-	-	-	-
60	1.00	0.182	1.54	0.185		-	-	-	-	-	-	-
73	1.30	0.241	1.54	0.248	0.538	0.328	3.50	0.84	1.32	0.86	0.64	0.84
76	1.41	0.244	1.54	0.222	0.536	0.336	4.00	0.96				
77	1.35	0.190	1.54	0.199	0.406	0.231	3.00	0.83	1.15	0.95	0.78	0.90
71	2.00	0.180	1.54	0.208	0.406	0.222	3.75	1.03	1.05	1.05	0.86	0.90
83	1.90	0.129	1.54	0.181	0.362	0.212	-	-	1.17	1.00	0.81	0.94
80	2.25	0.168	1.54	0.217	0.517	-	3.50	0.86	-	-	-	-
					Average		=	0.79	1.26	0.95	0.66	0.82

* Measured in the constant depth portion of the wave channel

JOURNAL

WATERWAYS DIVISION

Proceedings of the American Society of Civil Engineers

A RIPPLE TANK STUDY OF WAVE REFRACTION

G. C. Ralls, Jr.*
(Proc. Paper 911)

SYNOPSIS

The refraction of uniform waves on a beach of constant slope has been studied in a ripple tank to determine the effectiveness of Snell's Law for predicting wave refraction. The law was verified in this study by analyzing photographic records of wave refraction on a model beach. With this beach it was possible to vary beach slope and angle of orientation with respect to an oncoming wave train. The study covers three different slopes, 1:20, 1:40 and 1:60; three angles of orientation, 15°, 34°, and 50°; and several wave periods and water depths.

INTRODUCTION

When a wave moves into shoaling water its velocity of propagation decreases as the depth decreases, and continues to do so until the wave breaks. If waves approach the shore at an angle, the inshore portion of the wave front travels slower than the portion in deeper water and as a result the wave changes direction and swings around in such a manner that the crests tend to conform to the bottom contour. This phenomenon is known as wave refraction.

Application of the principles of wave refraction to the investigation of shore processes and harbor design is generally regarded as being essential to the proper understanding of those features of the problems involving wave action. Wave refraction coupled with direct shoaling governs the wave heights along a coast. The breaking of waves obliquely to the coast combined with agitation action heavily influences littoral sediment transport. Observations of wave refraction can also be used to estimate beach profiles where soundings cannot be taken. It is, therefore, of extreme importance that the theoretical basis of refraction theory be tested, and that its accuracy and limitations be determined.

Note: Discussion open until August 1, 1956. Paper 911 is part of the copyrighted Journal of the Waterways Division of the American Society of Civil Engineers, Vol. 82, No. WW 1, March, 1956.

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When considering a straight shore line and parallel offshore contours, wave refraction can be determined by applying Snell's Law (O'BRIEN, 1942):[4]

$$\frac{\sin \alpha_2}{\sin \alpha_1} = \frac{C_2}{C_1}$$

where α_1 and α_2 are angles between the wave crest and the bottom contour at any two points along the orthogonal to the crest, and C_1 and C_2 are the corresponding velocities of wave propagation at the points where α_1 and α_2 are considered.

Snell's law has been used in the past as the basis of wave refraction studies (CHIEN, 1954)[1] where the refraction of uniform and nonuniform waves on a beach of 1:40 slope and orientation of 15, 34, and 50 degrees was investigated. It is the purpose of this study to further that investigation by varying the slope; this will again present experimental evidence of the law's general applicability and possible limitations.

Laboratory Equipment

The study was conducted in a ripple tank (CHINN, 1949)[2] four feet wide, twenty feet long and five inches deep, which is shown in Figure 1. A 45 in. x 72 in. x 3/8 in. glass plate was installed in the tank bottom. A mirror oriented at 45 degrees with respect to the vertical, and below the glass section, reflected light from a carbon arc lamp used to approximate a point source of light. The light reflected from the mirror through the glass bottom, the transparent beach, the water surface, and onto a large shadow screen placed above the water surface. Light refracted through the wave crests and was focused on the screen, giving the exact shape of the wave. Motion pictures were taken of the shadows with a 35 mm camera and later analyzed for wave patterns.

The beach was made of 1/4 in. transparent lucite plate with toes of 18 gage galvanized iron. It was constructed to allow variation of slope and angle of orientation with respect to the oncoming wave train. Slopes used were 1:20, 1:40, and 1:60, with orientation angles of 15, 34, and 50 degrees. The beach extended the width of the tank in all cases. The toe of the beach was taped down to prevent movement. Black tape fastened to the beach was used to indicate a base-line and range-line. Plastic risers used for slope support provided a measuring grid.

A plunger-type wave generator, supported by a moveable frame, was located about five wave lengths from the toe of the beach. The wave period was controlled by changing the speed of the driving motor.

A wave absorber consisting of a piece of 1/4 in. plywood covered with burlap and a piece of expanded metal grating was placed at the downstream end of the tank to damp out any waves which reached that point, preventing interference with the pattern in the test section.

Laboratory Procedure

The beach was installed in the tank at a particular slope and orientation, and water was brought to the desired level. Still-water level was measured to within ±1/32 in. Several water depths were used to gain a large variation of water depths over the beach; the lowest level permitted the waves to break

on the beach. At subsequent levels water completely covered the beach. The beach under still water and a clock in motion were filmed separately to allow determination of the length and time scales of the photographic records. The wave generator was started, first using the slowest speed (longest wave period) which would focus waves on the shadow screen. After reaching steady state, the wave pattern was recorded on film taking pictures of about a dozen waves crossing the field of view. The motor speed on the generator was then increased (to produce a shorter wave period) and movies were again taken. About six periods were used, ranging from 0.15 to 0.6 seconds. After a series of runs at a particular water depth, the depth was increased, and the procedure repeated. After five 1/4-in. intervals of depth, the beach was re-oriented at a new angle, and the entire procedure was repeated. After data for three angles at one slope had been obtained, the beach slope was changed and the entire procedure repeated.

Method of Analysis

In using Snell's law to determine the position of wave crests at the surface, it is necessary to know the direction and velocity of the waves at a certain reference point.(1). This reference point generally is taken in deep water where the wave direction and velocity are constant. In this study it was taken at the toe of the beach beyond which the depth of water and the wave direction remained constant.

The films of the beach under still-water conditions were projected on a working table and the base line and range lines were drawn on a large sheet of clear celluloid for each angle of orientation. The grid on the beach determined the length scale of the projected film. The time scale was obtained by examining the movies of the clock in motion. The film of the wave pattern over the beach then was projected on the table and the celluloid used as an overlay, with base and range lines coinciding with those on the film. The actual crests were drawn on the overlay with a grease pencil which permitted easy erasure. Using the first few frames, a reference orthogonal was established. When the wave length was less than 0.33 feet, only one frame was used to establish the orthogonal; but when larger than this, two frames were used to more accurately define the orthogonal. This orthogonal line was checked with waves in following frames to see that it was consistant with the remainder of the run. All further measurements for the run were made along this orthogonal. The locations of the base-line, range-line, and reference orthogonal-line are shown in Figure 2, which is a frame indicating the pattern of 0.221 second waves over a 1:40 beach oriented at an angle of 34 degrees with the wave generator.

Each wave was numbered starting with time zero, and in subsequent projections these waves were identified by the same number. The distance, X, from the base line to the intersection of a given wave with the reference orthogonal (measured parallel to the range line) was recorded. The angle, α_2, which was the angle between the tangent to the wave at the point of measurement and the base line (same as that between crest and bottom contour) was recorded.

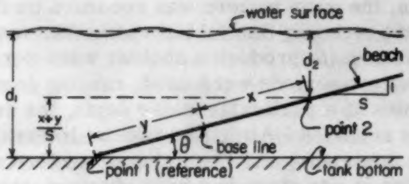
Knowing the period, T, the deep-water wave length, L_0 could be computed from the relationship:

$$L_0 = 5.12\ T^2.$$

Knowing the distance from the toe of the beach to the base line, Y, the distance from the base line to the measured point, X, the slope of the beach, S, and the still-water depth, d_1, the depth of water at the point of measurement, d_2, could be calculated from the relation:

$$d_2 = d_1 - \left(\frac{x+y}{s}\right)$$

θ small · assume $\cos\theta = 1$



which is explained by the accompanying sketch. Wave velocity was computed using the following relationship:

$$C = gT/2\pi \quad \tanh\ (2\pi d/L)\ .$$

Noting d_1 and L_o, the ratio d_1/L_o could be computed, from which tanh $(2\pi d/L_1)$ was obtained from published tables (WIEGEL, 1954). With this and the measured period, the velocity at constant depth of water, C_1, (at the toe of the beach and beyond) could be obtained. This value was constant for a particular run. Knowing d_2/L_o and the wave period, the wave velocity at the point of measurement, C_2, could be computed in a like manner. This value varied from point to point. The beach orientation angle, α_1, was an independent variable and hence, using Snell's Law, and the ratio C_2/C_1, the angle α_2 could be predicted.

Curves comparing the relationship between α_2 and d/L_o (Snell's Law) and measured values of α_2 are shown in Figures 3 to 9 inclusive.

Laboratory Results

Results obtained in this study fell into four general classes, designated classes A, B, C, and D, for purposes of discussion. In class A, the mean line of measured angles fell within ±1 degree of the curve predicted by Snell's law when α_1 was 34 and 50 degrees, and within ±1/2 degree when α_1 was 15 degrees. In class B, the mean line of measured angles was parallel to the curve predicted by Snell's Law, but deviated from that curve by a value greater than 1 degree. It is believed that this deviation could be due to possible inaccuracies in measurement of wave period. It was often difficult to accurately measure the period as a particular wave crest sometimes crossed the line of reference between frames of the film, despite the high film speed. The measurement was repeated to get an average value of T when this occurred, but each separate value of period deviated slightly from wave to wave. This could allow an error of 0.03 second in the period. Considering a typical case where α_1 = 34 degrees, T = 0.316 second, d_1 = 0.104 feet, and d_2 = 0.05 feet, if the measured period was 0.03 second too small, an error of 0.670 in α_2 predicted by Snell's Law would result. This error would be larger if α_1 was larger or if d_2 was smaller than the values chosen. In class C, the mean line of measured angles crossed the curve predicted by Snell's Law. In class D, the mean line of angles deviated from Snell's Law in a random manner,

but often approached a curve similar to the predicted one. It is believed that the deviations in classes C and D could be due to an error in orientation of the wave generator. In the laboratory system it was extremely difficult to orient the wave generator in a manner that would form a wave train exactly perpendicular to the walls of the tank (reference for orienting the beach). This error could reach the order of 3 degrees. It was often necessary to reorient the generator between runs for different speeds of the generator, as waves traveled in slightly different directions. Many of the runs which fell into classes C and D could be made to fall on a Snell curve for which α_1 was greater or less than the value used in the original calculations. In addition to the above error considerations, the general scatter of results could have been caused by the following laboratory or measurement errors.

1) The angle α_2 could be measured only to the nearest ±1 degree with estimation to the nearest ±1/2 degree.
2) Slight error could be introduced in the establishment of the reference orthogonal line. Though extreme care was used, it was not possible to fix the point of tangency to the wave exactly, and a possible deviation of 1 inch (actual size) could be common. This would make a difference in the calculation of d_2.
3) As the wave approached shore it often transformed into a small train of waves or broke. In deeper water the crests were less steep and did not focus light sharply. In both cases it was difficult to accurately define the position and direction of the wave crest.
4) In many runs the base line was not clear due to light refraction by the water surface undulation. This might have shifted the image of reference lines from their true position. In other words, the same wave in subsequent projections might not have been oriented with respect to the same base-line or range-line position. This would introduce an error either in establishment of the orthogonal or calculation of d_2.

Despite the above mentioned errors, analysis of the data indicated that the deviation of the measured values from Snell's Law was too consistent to be accounted for by experimental difficulties alone. The previous considerations of period measurement and generator orientation are felt to be most significant.

The runs were also classified into groups, ranging from 1 to 9. These groups were classed according to beach orientation and slope. Groups 1, 2 and 3 had beach slopes of 1:60 with orientations of 15, 34 and 50 degrees, respectively. Groups 4, 5 and 6 had beach slopes of 1:40 and orientations of 50, 34 and 15 degrees, respectively. Groups 7, 8 and 9 had beach slopes of 1:20 and orientations of 15, 34,and 50 degrees, respectively.

Typical Snell curves and corresponding measured angles for the four classes in group 1 are shown in Figure 3. 24 percent of the runs fell into class A, 27 percent into class B, 11 percent into class C, and 38 percent into class D. Average deviation from each run in all groups is given in Table 1. The maximum deviation for group 1 (based on the average of 1 run) was 9 degrees with the average absolute deviation (taken without respect to whether deviation was positive or negative) of all runs in this group was 2.23 degrees and the average deviation (considering whether deviation was positive or negative was -1.75 degrees. Typical results for group 2 are shown in Figure 4. 32 percent of the runs fell into class A, 34 percent into class B, 0 percent into class C, and 34 percent into class D. Maximum deviation was 11 degrees, with average absolute deviation 2.56 degrees and average deviation -2.27

degrees. Typical results for group 3 are shown in Figure 5. 18 percent of the runs fell into class A, 43 percent into class B, 36 percent into class C, and 3 percent into class D. Maximum deviation was 15 degrees with average absolute deviation 4.52 degrees and average deviation 3.16 degrees. Typical results for group 4 are shown in Figure 6. 48 percent of the runs were in class A, 22 percent in class B, 8 percent in class C, and 22 percent in class D. Maximum deviation was 8 degrees with average absolute deviation of 1.90 degrees and average deviation of -0.66 degrees. Typical results for group 5 are shown in Figure 7. 41 percent of the runs were in class A, 38 percent in class B, 14 percent in class C, and 7 percent in class D. Maximum deviation was 4 degrees with average absolute deviation 1.36 degrees and average deviation 0.16 degrees. Many of the runs in groups 6, 7, 8 and 9 were in the "deep water" case (i.e., $\alpha_2 \cong \alpha_1$ at $d_2/L \geq 0.40$). As a result, several deep water-cases in a particular group were plotted against the same Snell curve rather than using several different curves to illustrate the classes into which the runs fell. The classes can generally be seen from the points shown compared to the one curve. The curve for group 6 is shown in Figure 8. 32 percent of the runs were in class A, 14 percent in class B, 36 percent in class C, and 18 percent in class D. Maximum deviation was 5 degrees with average absolute deviation 1.76 degrees and average deviation +0.53 degrees. The curve for group 7 is shown in Figure 8. 41 percent of the runs fell into class A, 14 percent into class B, 14 percent into class C, and 31 percent into class D. Maximum deviation was 3 degrees with average absolute deviation 1.01 degrees and average deviation -0.73 degrees. The curve for group 8 is shown in Figure 9. 40 percent of the runs were in class A, 20 percent were in class B, 15 percent in class C, and 25 percent in class D. Maximum deviation was 2.8 degrees with average absolute deviation 1.37 degrees and average deviation -1.26 degrees. The curve for group 9 is shown in Figure 9. Maximum deviation was 3.5 degrees with average absolute deviation 1.76 degrees and average deviation -0.50 degrees.

Discussion of Results

Considering groups 1 through 5 (Figures 3-7) little can be seen from the figures presented in this paper as to the overall effectiveness of Snell's Law as only a few measured angles could be compared to a particular Snell curve. It can be noted from the figures that the points appear to follow the general trend of Snell's Law. In all groups, over 50 percent of the runs were in classes A and B. In both these classes the mean line of points exactly followed the trend predicted by Snell's Law, class B deviating by a constant amount. Many runs in classes C and D were made to fall on the curve predicted by Snell's Law by using a different α_1 on which to base calculations. The results in groups 1 through 5 do indicate the probable accuracy of Snell's Law.

Groups 6 through 9 (Figures 8 and 9) indicate more concretely the accuracy of Snell's Law because many runs could be compared with a single curve and a definite trend of measured points established. The agreement is self-evident, but it might be noted that the centroid of the point groupings falls a little below the predicted curve for values of d_2/L larger than 0.25 in groups 7 and 8. This deviation might be due to any of the previously mentioned laboratory and/or measurement errors. In both cases the order of magnitude of deviation was about 2 to 3 degrees. This deviation is not attributed to inaccuracy

of Snell's Law in this region as it did not occur frequently enough to establish a trend in all groups.

In conclusion, this study essentially verified Snell's Law on refraction of uniform, long-crested waves in the region $0.10 \leq d_2/L \leq 0.70$.

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REFERENCES

1. CHIEN, N., Some ripple tank studies of wave refraction, Trans. American Geophysical Union, vol. 35, no. 6, pp 897-904, Dec., 1954.
2. CHINN, A. J., The effect of surface tension on wave velocities in shallow water, M. S. Thesis, University of California, Berkeley, Calif., 1949.
3. JOHNSON, J. W., O'BRIEN, M. P., and ISAACS, J. D., Graphical construction of wave refraction diagrams, U.S. Hydrographic Office, H.O. Pub. No. 605, January 1948.
4. O'BRIEN, M. P., A summary of the theory of oscillatory waves, Beach Erosion Board, Technical Report No. 2, 1942.
5. WIEGEL, R. L., Gravity waves; tables of functions. Council on Wave Research, the Engineering Foundation, University of California, Berkeley, February 1954.

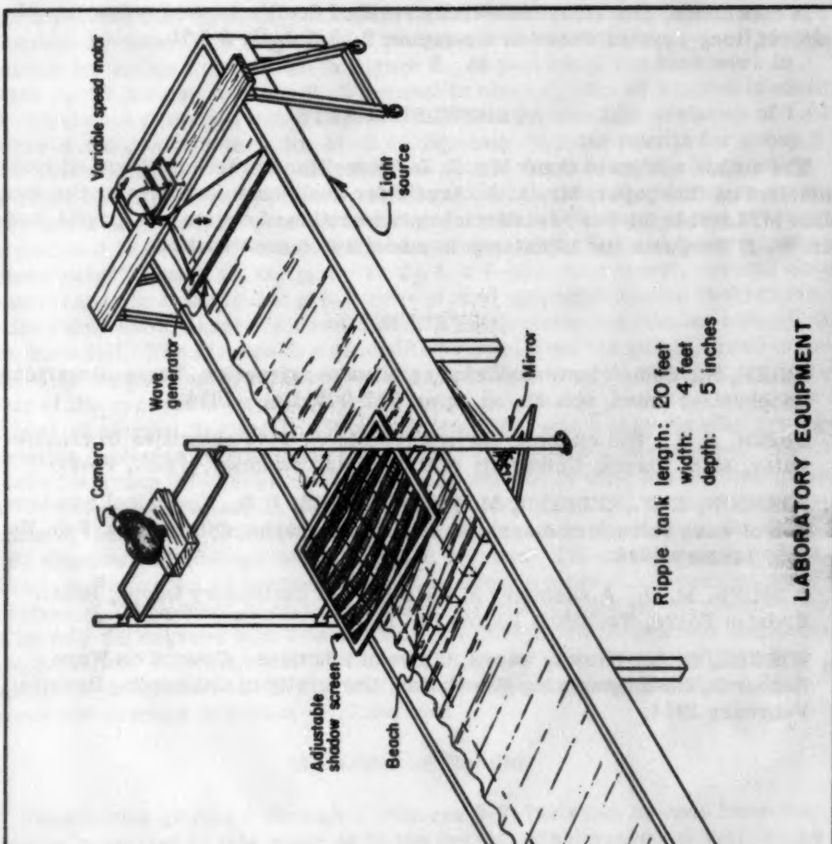

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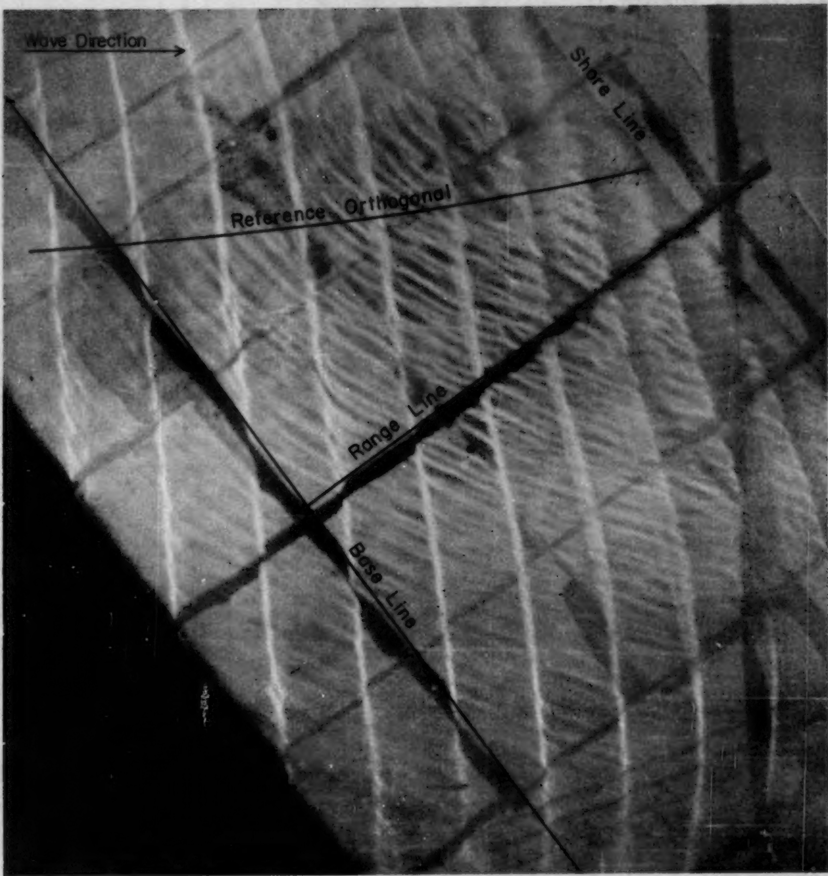
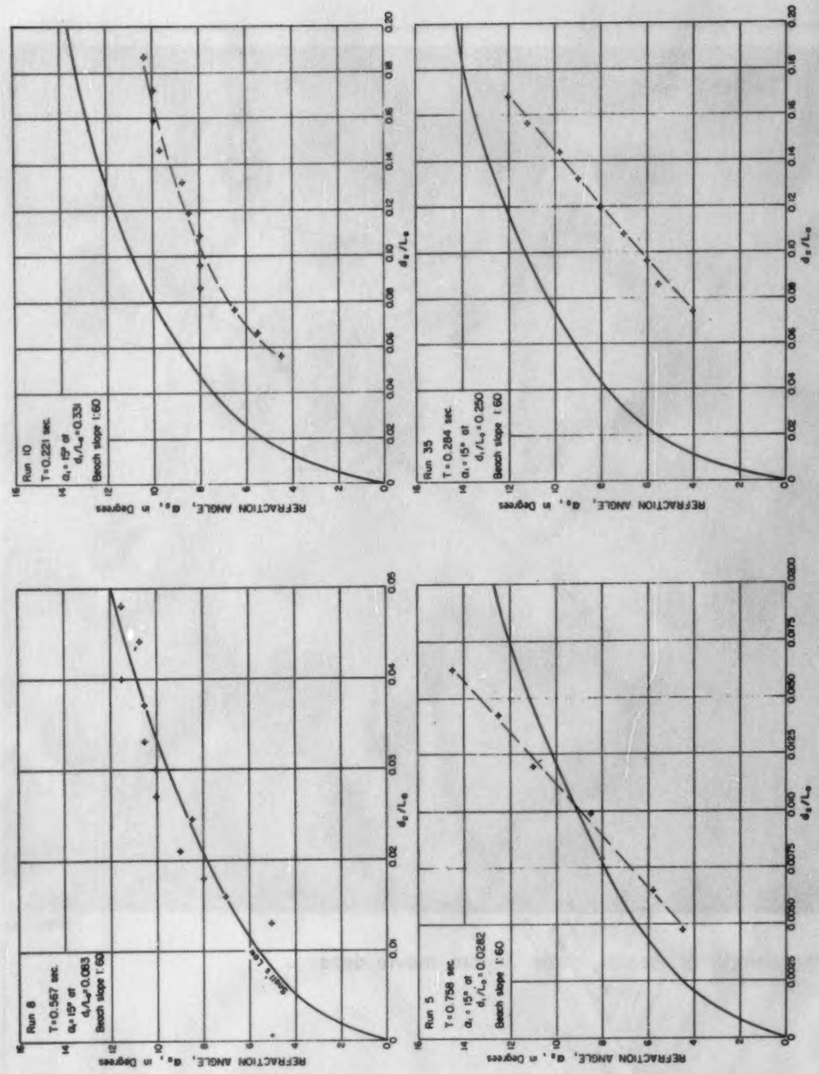
FIGURE 1



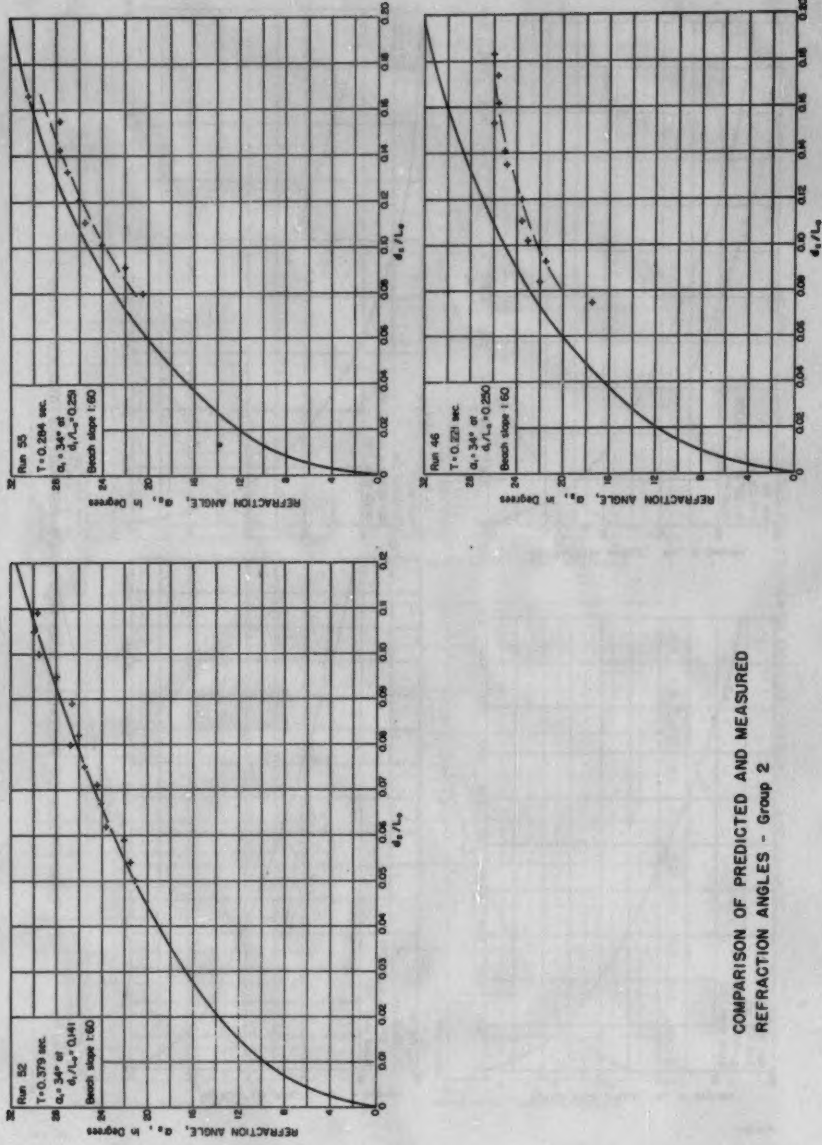
HYD-7166

Photograph of Beach, from 35mm movie data



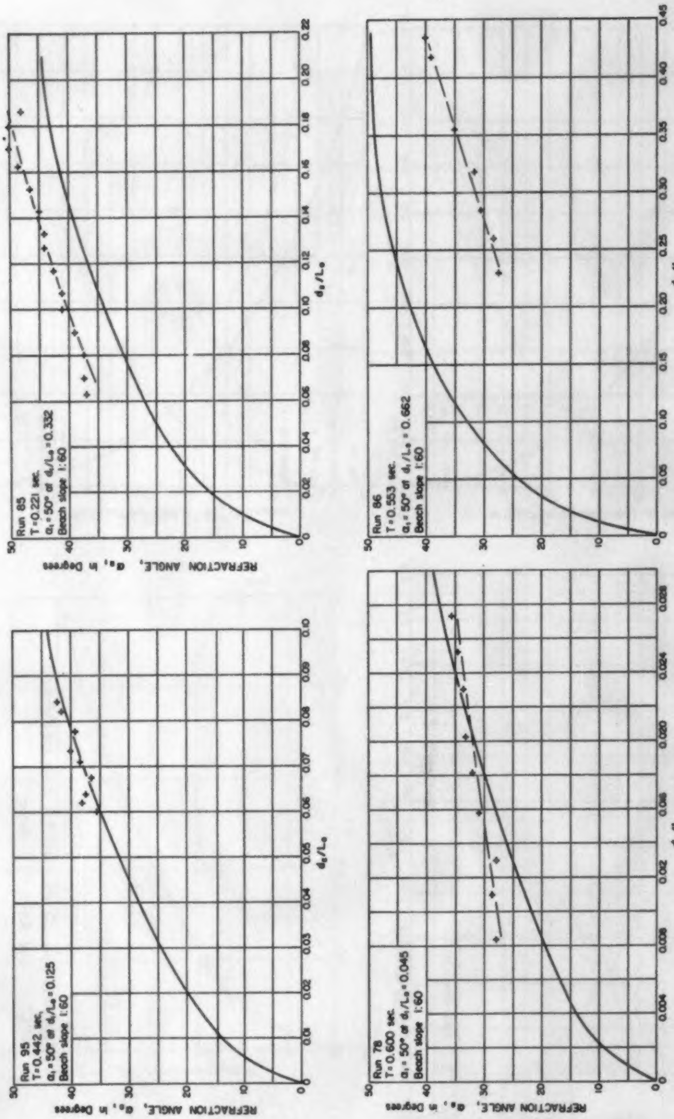
COMPARISON OF PREDICTED AND MEASURED REFRACTION ANGLES - Group I

FIGURE 3



COMPARISON OF PREDICTED AND MEASURED REFRACTION ANGLES - Group 2

FIGURE 4



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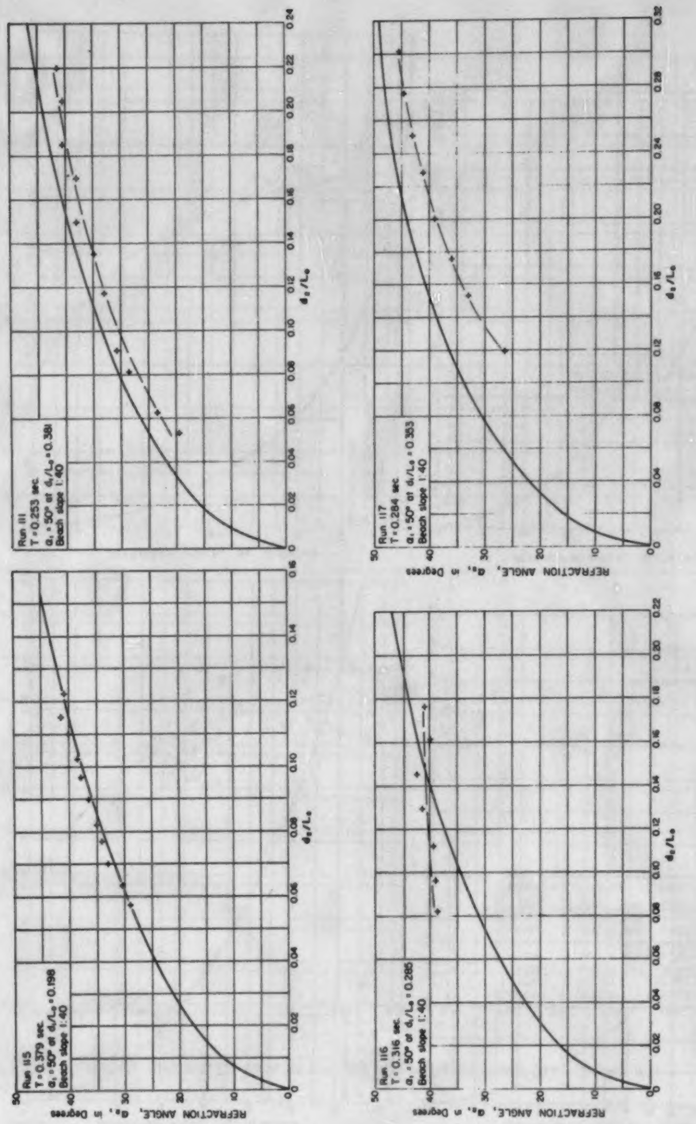
COMPARISON OF PREDICTED AND MEASURED REFRACTION ANGLES - Group 3

FIGURE 5



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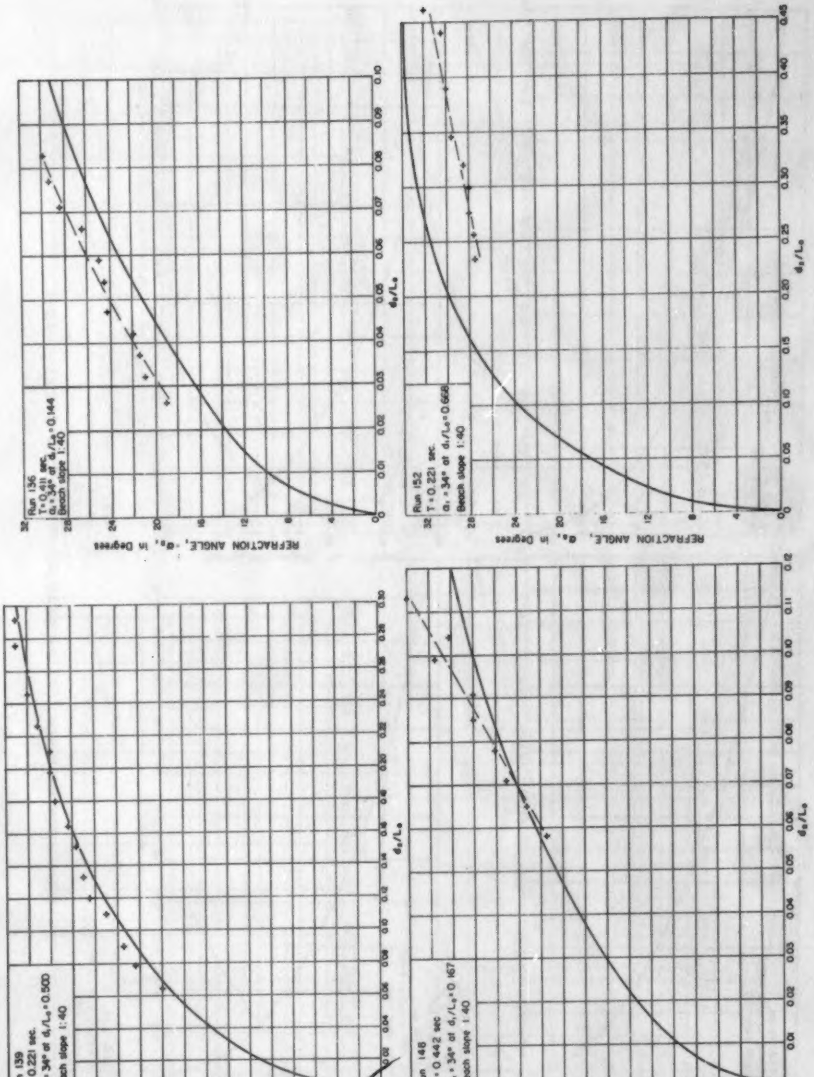
COMPARISON OF PREDICTED AND MEASURED REFRACTION ANGLES - Group 4

FIGURE 6



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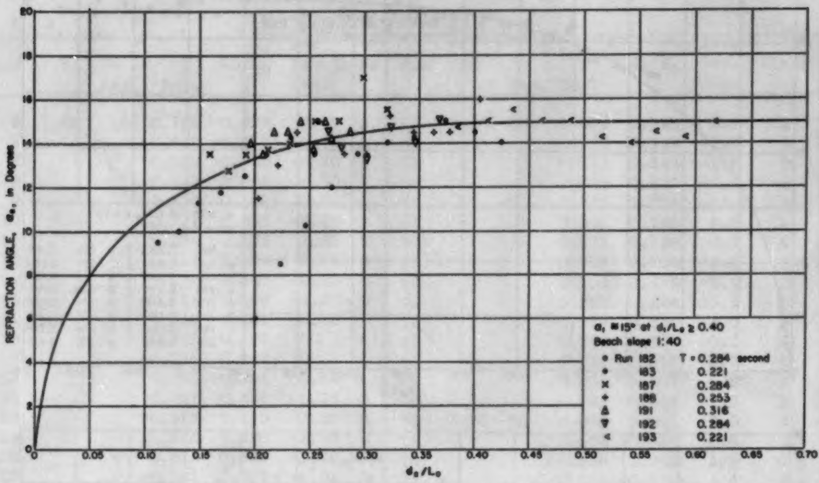
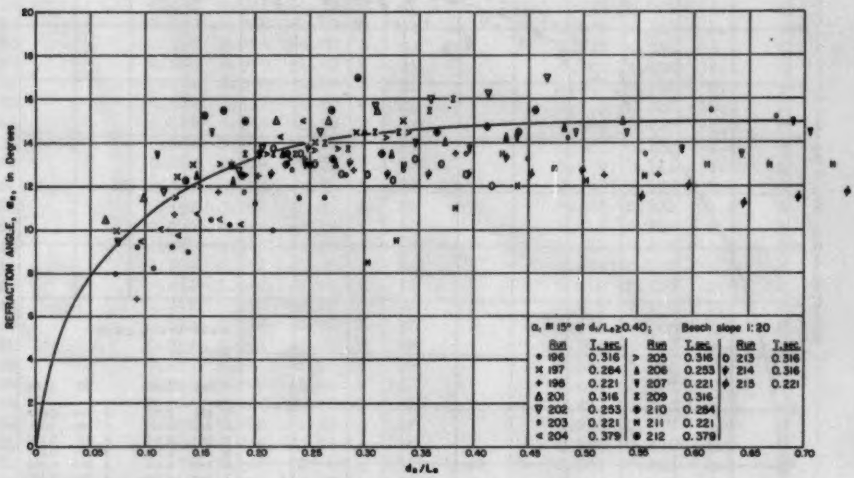
COMPARISON OF PREDICTED AND MEASURED REFRACTION ANGLES - Group 5

FIGURE 7





COMPARISON OF PREDICTED AND MEASURED REFRACTION ANGLES - Groups 6 & 7

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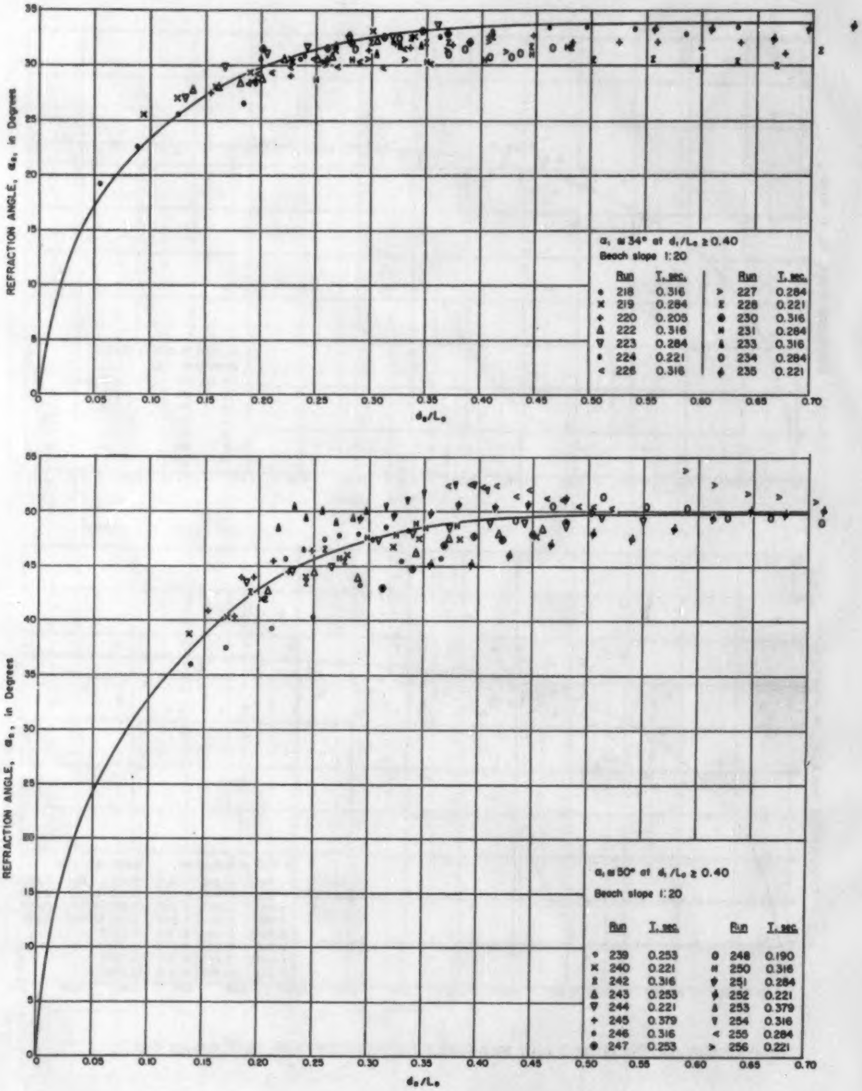
FIGURE 8



COMPARISON OF PREDICTED AND MEASURED REFRACTION ANGLES - Groups 8 & 9

HYD-7173

FIGURE 9

TABLE I.

EXPERIMENTAL RESULTS

Run	S	α_1 Deg.	T Sec.	d_1/L_o	Ave. Dev. Deg.	Class
5	1:60	15	0.758	0.028	0.50	C
6			0.663	0.037	0.75	A
7			0.569	0.050	-0.20	A
8			0.442	0.083	0.20	A
9			0.316	0.162	-0.20	A
10			0.221	0.331	-3.10	B
11			0.474	0.109	1.00	B
12			0.411	0.144	2.50	B
13			0.316	0.247	-2.70	D
14			0.221	0.498	-2.30	D
15			0.348	0.201	-2.50	D
18			0.379	0.198	0	A
19			0.316	0.788	0	A
20			0.248	0.362	0.30	A
21			0.221	0.582	-2.10	D
23			0.427	0.179	-1.80	B
24			0.348	0.270	-2.50	B
25			0.316	0.326	-2.50	D
26			0.253	0.509	-5.00	D
27			0.221	0.665	-8.80	D
28			0.632	0.051	-0.50	C
29			0.600	0.056	-1.20	D
30			0.537	0.070	-4.00	B
31			0.506	0.079	-1.60	B
32			0.474	0.090	-2.00	D
33			0.379	0.142	-1.40	C
34			0.348	0.168	-3.50	D
35			0.284	0.250	-4.80	D
36			0.221	0.414	-6.80	B
37	1:60	34	0.696	0.034	-1.90	D
38			0.664	0.037	-0.50	A
39			0.600	0.450	-2.30	B
40			0.569	0.050	-4.00	B
41			0.474	0.072	-1.00	A
42			0.442	0.084	-2.30	B
43			0.379	0.113	-3.30	D
44			0.348	0.134	-1.50	D
45			0.284	0.201	-2.00	D
46			0.221	0.250	-3.50	D
48			0.664	0.046	-2.50	D
49			0.600	0.057	-2.20	D
50			0.569	0.036	-1.20	B
51			0.506	0.079	-0.70	A
52			0.379	0.141	0	A
53			0.411	0.120	-0.40	A
54			0.316	0.203	-0.40	A
55			0.284	0.251	-1.20	B
56			0.221	0.416	-5.00	B
58			0.474	0.109	-1.00	A
59	1:60	34	0.411	0.145	0.0	A
60			0.379	0.169	-1.5	B
61			0.316	0.244	-1.5	B
62			0.284	0.302	-1.5	B
63			0.190	0.675	-11.0	B
65			0.442	0.145	0.0	A
66			0.379	0.198	0.0	A
67			0.316	0.285	2.5	B
68			0.284	0.438	-8.0	D
69			0.221	0.584	-10.0	D
71			0.379	0.227	-4.0	D
72			0.379	0.227	0.0	A
73			0.316	0.326	2.3	B
74			0.284	0.404	-2.0	D
75			0.221	0.668	-7.3	D
76	1:60	50	0.632	0.041	4.0	C
77			0.664	0.037	2.0	B
78			0.600	0.045	3.0	C
79			0.569	0.050	4.5	B
80			0.569	0.050	10.6	B
81			0.442	0.083	4.5	B
82			0.379	0.113	3.5	B
83			0.348	0.134	5.5	C
84			0.284	0.200	6.2	C
85			0.221	0.332	6.0	B
86			0.553	0.662	-15.2	D
87			0.474	0.040	-1.0	C
88			0.442	0.232	-3.0	B
89			0.378	0.141	1.0	A
90			0.348	0.167	2.5	B
91			0.316	0.203	1.0	B
92			0.316	0.203	2.5	B
93			0.221	0.416	2.5	B
94			0.474	0.109	1.3	C
95			0.442	0.125	0.0	A
96			0.379	0.170	1.5	B
97			0.316	0.244	1.0	A
98			0.284	0.302	5.0	C
99			0.253	0.381	6.5	C
101			0.442	0.167	0.0	A
102			0.411	0.193	0.0	A
103			0.348	0.269	1.3	C
104			0.284	0.404	2.0	C
106	1:40	50	0.506	0.095	0.0	A
107			0.474	0.108	0.0	A
108			0.411	0.144	0.0	A
109			0.379	0.170	1.5	B
110			0.316	0.244	0.0	A
111			0.253	0.381	-2.6	B

Table 1 continued.

Run	S	α_1 Deg.	T Sec.	d_1/L_o	Ave. Dev. Deg.	Class
112	1:40	50	0.190	0.675	-4.3	D
113			0.506	0.111	0.0	A
114			0.442	0.146	0.8	A
115			0.379	0.198	0.5	A
116			0.316	0.285	0.0	C
117			0.284	0.353	-5.0	D
118			0.221	0.584	-7.0	D
119			0.537	0.113	-0.5	A
120			0.379	0.227	-0.7	A
121			0.316	0.326	0.0	A
122			0.284	0.404	0.0	A
123			0.221	0.668	8.6	D
124			0.442	0.188	0.0	A
125			0.379	0.255	1.0	A
126			0.348	0.303	0.5	C
127			0.253	0.573	-2.5	D
128			0.221	0.752	-3.8	B
129			0.379	0.282	-2.5	B
130			0.316	0.407	-2.2	B
131			0.284	0.503	-3.3	D
132			0.253	0.634	-4.5	B
133	1:40	34	0.569	0.075	2.8	D
134			0.474	0.109	0.7	A
135			0.442	0.125	2.2	B
136			0.411	0.144	3.0	B
137			0.316	0.244	1.0	A
138			0.284	0.302	1.3	B
139			0.221	0.500	0.6	A
140			0.506	0.111	1.0	A
141			0.474	0.127	2.0	B
142			0.427	0.156	1.2	B
143			0.379	0.198	1.0	C
144			0.348	0.235	0.0	A
145			0.284	0.353	0.6	A
146			0.221	0.584	7.0	B
147			0.506	0.127	1.4	B
148			0.442	0.167	1.2	C
149			0.379	0.227	-4.0	B
150			0.316	0.326	0.0	A
151			0.284	0.404	-0.5	C
152			0.221	0.668	-4.0	D
154			0.411	0.217	0.0	A
155			0.379	0.255	0.0	A
156			0.316	0.367	0.0	A
157			0.284	0.455	-0.5	A
158			0.253	0.573	-3.0	B
160			0.379	0.282	-0.8	C
161			0.316	0.407	0.0	A
162			0.284	0.503	-2.5	B
163			0.253	0.573	-2.0	B
164	1:40	15	0.600	0.068	-5.0	D
166			0.474	0.109	-2.1	A
167			0.442	0.125	0.2	A
168	1:40	15	0.379	0.170	-1.2	B
169			0.316	0.244	0.7	A
170			0.284	0.302	1.3	B
171			0.253	0.328	2.7	B
172			0.458	0.156	1.8	C
179			0.442	0.167	-2.8	C
180			0.379	0.227	-0.5	C
181			0.316	0.326	1.2	D
182			0.284	0.404	-1.3	C
183			0.221	0.668	-3.2	C
184			0.442	0.188	-3.6	D
185			0.379	0.255	0.7	C
186			0.316	0.367	0.5	C
187			0.248	0.455	0.3	A
188			0.253	0.573	-0.7	D
190			0.348	0.335	-2.0	C
191			0.316	0.407	-0.5	A
192			0.284	0.503	-0.2	A
193			0.211	0.832	0.3	A
194	1:20	15	0.442	0.250	0.0	A
195			0.379	0.340	0.0	A
196			0.316	0.489	-2.5	D
197			0.284	0.605	0.2	A
198			0.221	1.000	-1.0	A
199			0.442	0.271	-1.5	C
200			0.379	0.368	-0.7	C
201			0.316	0.530	0.0	D
202			0.253	0.836	0.5	D
203			0.221	1.080	-1.2	B
204			0.379	0.397	-1.9	C
205			0.316	0.571	-0.3	A
206			0.253	0.890	-0.7	A
207			0.221	1.170	-0.7	A
208			0.411	0.361	0.0	A
209			0.316	0.612	0.0	A
210			0.284	0.757	-0.9	D
211			0.221	1.250	-2.5	D
212			0.379	0.453	2.0	B
213			0.333	0.651	-2.2	D
214			0.316	0.651	-1.5	D
215			0.221	1.330	-3.0	B
216	1:20	34	0.537	0.169	1.0	A
217			0.340	0.340	-2.8	D
218			0.316	0.489	0.0	A
219			0.284	0.605	-0.6	A
220			0.205	1.146	-1.5	C
221			0.379	0.340	-0.5	A
222			0.316	0.528	-1.0	C
223			0.284	0.658	-0.2	A
224			0.221	1.080	-0.5	A
225			0.379	0.397	-2.6	B
226			0.316	0.571	-2.3	D
227			0.284	0.707	-1.3	D

Table 1 continued.

Run	S	α_1 Deg	T Sec.	d_1/L_o	Ave. dev. Deg.	Class
228	1:20	34	0.221	1.168	-3.8	B
229			0.379	0.425	-1.0	C
230			0.316	0.612	0.0	A
231			0.284	0.758	-2.3	B
232			0.221	1.250	-2.4	D
233			0.316	0.651	-1.2	B
234			0.284	0.806	-2.5	D
235			0.221	1.332	-0.9	A
236	1:20	50	0.442	0.250	-3.0	D
237			0.379	0.340	-3.5	D
238			0.316	0.489	-2.0	B
239			0.253	0.762	-4.0	D
240			0.221	1.000	-2.0	D
241			0.379	0.369	0.0	A
242			0.316	0.528	-1.3	B
243			0.253	0.823	-2.7	B
244			0.221	0.080	-0.5	A
245			0.379	1.125	0.5	A
246			0.316	0.547	1.0	A
247			0.253	0.823	-1.5	D
248			0.190	1.463	0.0	A
249			0.379	0.425	-0.2	A
250			0.316	0.612	0.0	A
251			0.284	0.757	1.0	A
252			0.221	1.250	-3.0	D
253			0.379	0.453	3.5	B
254			0.316	0.651	3.2	B
255			0.284	0.806	2.0	D
256			0.221	1.332	2.0	D

JOURNAL

WATERWAYS DIVISION

Proceedings of the American Society of Civil Engineers

DESTRUCTION OF WAVE ENERGY BY VERTICAL WALLS

Per Bruun[a]
(Proc. Paper 912)

INTRODUCTION

Wave energy can be destroyed or affected in different ways by: diffraction, refraction, reflection, breaking, resonance, friction, permeability, special hydraulic arrangements as stilling basins and berms, and by air-entrainment. A summary is given of the different methods of destroying or affecting wave energy.

Destruction of Wave Energy in Harbors

Diffraction - The first ships were built for use in streams, fiords and bays where only small waves occurred. When the dimensions of the ships were increased and vertical walls became necessary for loading or unloading the ships, the engineering problem in harbor technology started. Big cities, e.g. Rome, founded on the riverside, needed harbors for seagoing vessels which had to be constructed near the city, often in a coastal area where there was no natural harbor and where the area had to be protected against waves and littoral drift by jetties or breakwaters. The harbor at Ostia was constructed to serve Rome but the jetties were not able to protect the harbor against littoral drift and it now lies some two miles inland.

At Ostia, as at many other harbors, there was constructed recently an outer harbor and an inner harbor, see Fig. 1a. In the outer harbor the wave height is decreased by wave diffraction and only a small part of the wave energy in the outer harbor passes into the inner harbor. See (9) and (11).

Refraction - Refraction is used in different ways for calming waves in harbors. Some harbors have approach-canals and the waves are refracted along the sides of these canals, and other harbors have beaches within towards which the waves are refracted. See Fig. 1b. A great many of the harbors are built in bays and fiords where refraction is taken advantage of to the utmost extent. See (11).

Reflection - At some harbors the jetties are turned outward as shown in Fig. 1c in order to reflect wave energy in a direction which does not create a hazard to navigation. Reflection is also used with submerged and floating breakwaters. See (4), (6), (7), (10), (16), and (19).

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Breaking - Sometimes harbors are constructed behind a shoal or an island in the sea. See Fig. 1d. The shoal destroys wave energy by refraction and breaking of waves. Many harbors have a beach or some other kind of gentle slope in the harbor for destroying wave energy by breaking the waves and/or decreasing the amount of energy reflected. See (25).

Resonance - A new but very interesting principle mentioned in (23) is destruction of wave energy by resonance. Wave energy is absorbed by means of basins built on both sides of the harbor mouth with dimensions as indicated in Fig. 1e where L_1, L_2 and L_3 are different occurring wave lengths. These basins can be developed in different ways as described in (23).

This type of resonance differs entirely from the very troublesome resonance which appears in some harbor basins (Seiche and Surge.) See (1) and (24).

Friction - Wave energy can be absorbed by friction, e.g. sleepers can be put on the bottom or different kinds of ruggedness can be arranged on slopes where uprush takes place. Friction arrangements on the bottom of a harbor are not practical but in flood control technology they are in common use; friction on slopes can be produced by rubble mounds by means of sleepers or prisms, or by gentle slopes which may be provided with wide berms (Fig. 1f). See (3) and (18).

Permeability - Permeability is taken advantage of in rubble mounds, rock-filled breakwaters and in other kinds of permeable constructions, for example fascine walls. See Fig. 1g. The efficiency of permeability increases to a certain limit as the volume of voids in the entire permeable structure is increased. Permeability is especially valuable when the slope is steeper than 1 on 1.5. See (3) and (17).

Special hydraulic arrangements - Such arrangements are used in the form of hydraulic stilling basins or berms put in a slope which may be permeable or impermeable and in the form of submerged or floating breakwaters. (Fig. 1h). See (3), (8), and (10).

Air entrainment (pneumatic breakwaters) - Destruction of wave energy by air-bubbles from a pipeline situated on the bottom of the sea (Fig. 1i) has been used with good luck in California (21) and in England (22). Pneumatic breakwaters provide an air lift whereby a steady stream of water is maintained and the oscillating movement in the wave is slowed down.

Destruction of Wave Energy by Dividing a Reflected Wave into Separate Parts of Different Phases

The following section describes how the incident wave is divided into two parts which are reflected with a phase difference, e.g. 180 degrees. See Fig. 2. This means that one part of the reflected wave has a crest when the other part has a trough, consequently strong transversal currents creating much loss of energy will come into existence. Meanwhile there is the problem of how such conditions can be created. Fig. 3 shows two walls, the front wall permeable and the back wall impermeable. The space between the two walls (front sides) is shown by 'a.' Thickness of the wall is small compared to the other dimensions.

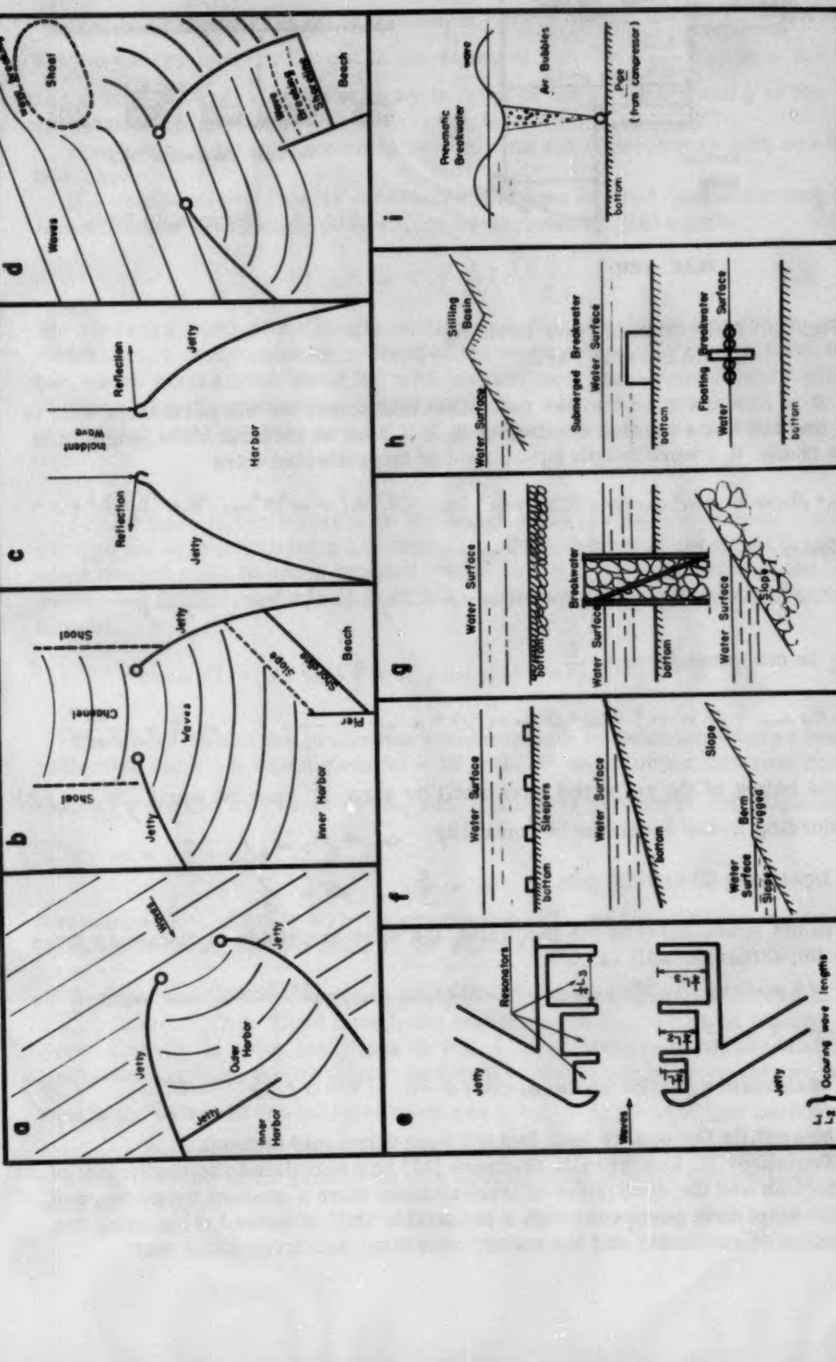

Fig. 1. Destruction of Wave Energy as Practiced in Harbor Engineering.

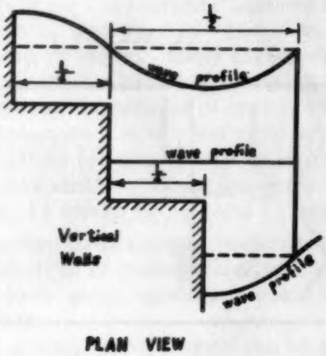

Fig. 2. Reflection of Wave Dividing it into Several Parts.

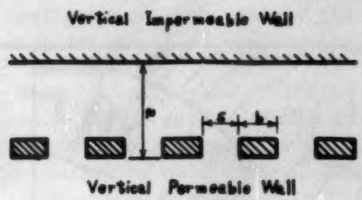

Fig. 3. Permeable Wall.

It is now assumed that the reflection coefficient for the permeable wall is β and the transmission coefficient is α. With an incident wave height of H the phase θ_1, wave length L_1 height of the reflected wave

$$H_R = \beta \cos\theta + \alpha^2 \cos\left(\theta + \frac{4\pi a}{L}\right) + \alpha^2\beta \cos\left(\frac{8\pi a}{L}\right) + \alpha^2\beta^2 \cos\left(\theta + \frac{12\pi a}{L}\right) + ---$$

When $\theta = 0$

$$H_R = \beta + \alpha^2 \cos\left(\frac{4\pi a}{L}\right) + \alpha^2\beta \cos\left(\frac{8\pi a}{L}\right) + \alpha^2\beta^2 \cos\left(\frac{12\pi a}{L}\right) + ---$$

H_R is minimum for $a = \frac{L}{4}$

$$H_{R\,min.} = \beta - \alpha^2 + \alpha^2\beta - \alpha^2\beta^2 + ------ \qquad = \beta - \frac{\alpha^2}{1+\beta}$$

If the height of the reflected wave shall be zero, β must be equal $\frac{\alpha^2}{1+\beta}$ (I)

According to the equations of continuity $\alpha + \beta = 1$ (II)

Equations (I) and (II) give $\alpha = \frac{2}{3}$, $\beta = \frac{1}{3}$

In the space between the two walls, the wave amplitude in distance x from the impermeable wall is

$$\alpha \cos\theta + \alpha \cos\left(\theta + \frac{4\pi x}{L}\right) + \alpha\beta \cos\left(\theta + \frac{4\pi a}{L}\right) + \alpha\beta \cos\left(\theta + \frac{2\pi(2a+2x)}{L}\right) + \alpha\beta^2 \cos\left(\theta + \frac{8\pi a}{L}\right) + --$$

When $\frac{a}{L} = \frac{1}{4}$

the maximum agitation in the basin $(\theta = 0,\ x = 0)$ is $\frac{2\alpha}{1+\beta}$ (III)

Meanwhile the energy loss has not been taken into account.

Professor H. Lundgren in Denmark (13) has calculated the coefficient of reflection and the coefficient of transmission when a shallow water wave of small amplitude passes through a permeable wall of spaced piles using the equation of continuity and the energy equation. Lundgren found that

$$\beta = \frac{kH}{3\pi d}(1-\beta)^2$$

when β is the coefficient of reflection, H = wave height, d = water depth and k = the energy loss coefficient in the equation $\Delta h = \frac{kv^2}{2g}$, when Δh is the loss in head, v is the velocity in front of the structure and g is the acceleration of gravity.

Concerning k no data seems to be available for experiments with waves in this case.

If a unidirectional flow is considered, the loss of head Δh according to investigations carried out in Germany by Kirschener (20) equals

$$\Delta h = k'\left(\frac{s}{b}\right)^{\frac{4}{3}} \frac{v^2}{2g}$$

when s is the width of the single obstruction, b is the space between the obstructions, v is the velocity in frcnt of the obstruction, and k^1 is a form factor, which varies from about 0.8 with streamlined (fish-shaped) obstructions to about 2.5 with rectangular obstructions.

If Lundgren's formula is used with s = b, k = 2.5 and $\frac{H}{d}$ = 0.2 we find

$$\beta \sim 0.05$$

Preliminary experiments with reflection from permeable vertical walls carried out at the Hydraulic Laboratories, Copenhagen, showed that k for wave motion must be much greater, which is not surprising because the oscillating motion of water in the waves is entirely different to unidirectional motion.

When β = 1/3 and $\frac{H}{d}$ = 0.2 we find k = $k'\left(\frac{s}{b}\right)^{\frac{4}{3}} = 35$

The above mentioned preliminary experiments in Denmark showed least reflection for s = b which gives k^1 = 35. The k^1 value under different conditions of wave motion, water depth, and $\frac{s}{b}$ ratio is now under investigation.

If $\alpha = \frac{2}{3}$ and $\beta = \frac{1}{3}$

and comparison is made with the expression (III), we find that the maximum agitation between the two walls is the same as the height of the incident wave.

In experiments carried out in France (12), the construction showed in Fig. 4 is mentioned. Even though the conditions in Fig. 3 cannot be compared directly with the conditions in Fig. 4, it is interesting to note that the permeability, when the maximum agitation in the space between the two walls equals the height of the incident wave and a = $\frac{L}{4_1}$ is 30 to 50 per cent which confirms the results in Denmark and shows that the loss coefficient $k^1 (\frac{s}{b})$ 4/3 = k, must be much greater in this special case of wave motion than with unidirectional flow.

In connection with the above mentioned, credit should also be given to Biesel's theoretical and experimental work in France and Costello's experiments in California. See e.g. Project Report No. 38 from St. Anthony Falls Laboratory, University of Minnesota, 1953.

Use of Phase Displacement of Reflected Waves

Harbor Technology - Fig. 5 shows a harbor. Wave energy passes in between the jetties with part of the energy being reflected from the jetties toward the inner parts of the harbor. If the jetties have vertical walls, 100 per cent of the energy will be reflected. With sloping walls only a part of the energy will be reflected. Meanwhile any energy reflected means trouble. Measures taking the above-mentioned principle into consideration are shown in Fig. 6. The distance between the reflecting slopes is

$$\frac{L}{4} + n \cdot \frac{L}{2} \quad (n = 1, 2, 3, \text{---})$$

when L is the wave length for the most troublesome period. Because of the 180 degrees phase displacement, transversal currents will come into existence and energy will be destroyed. A minor amount of energy will be reflected towards the harbor mouth. If the distance between the reflecting slopes is less or greater than $\frac{L}{4} + n \cdot \frac{L}{2}$, less energy will be destroyed but it will still be reflected toward the harbor mouth and not into the harbor even if part of it will be diffracted in the outer harbor.

Fig. 7 shows a photograph of an experiment carried out in Copenhagen with a Naval (NATO) harbor which is now under construction in the Danish seas. The investigations were made with different directions of wave propagation, wave length, and wave height. The result of the investigations indicated that the wave traps decreased the wave height in the inner harbor considerably.

These wave traps can be constructed as rubble mounds separated with pile walls which need not be absolutely water-tight but the permeability should be rather low. The dimensions of the wave traps are given by the angle between the reflecting jetty and the direction of wave propagation through the mouth of the harbor, and the wave length. See Fig. 6.

Coastal Protection Technology - Destruction of wave energy is very important in littoral drift and coastal protection technology. Investigations of littoral drift caused by wave motion carried out at the University of California (14) give some reason to believe that bed-load transportation depends on the maximum velocity raised to about the 7th power (solitary waves in comparatively shallow water).

In coastal protection technology we usually deal not with oscillating waves but with surf, which is a solitary wave with an unsymmetric shape, steep front side and gentle back side. Vertical permeable walls reflect such waves 100 per cent, after which they travel back over the beach, stirring up the material a second time. It is therefore very important to destroy the energy in the surf as much as possible in order to prevent the removal of beach material caused by the reflected surf and longshore currents. A permeable wall is able to reflect the surf in small parts and at the same time can destroy wave energy so that the water flows back instead of being reflected as an entire wave. Meanwhile the destruction of the energy is accompanied by much turbulence and beach material is stirred up. It is

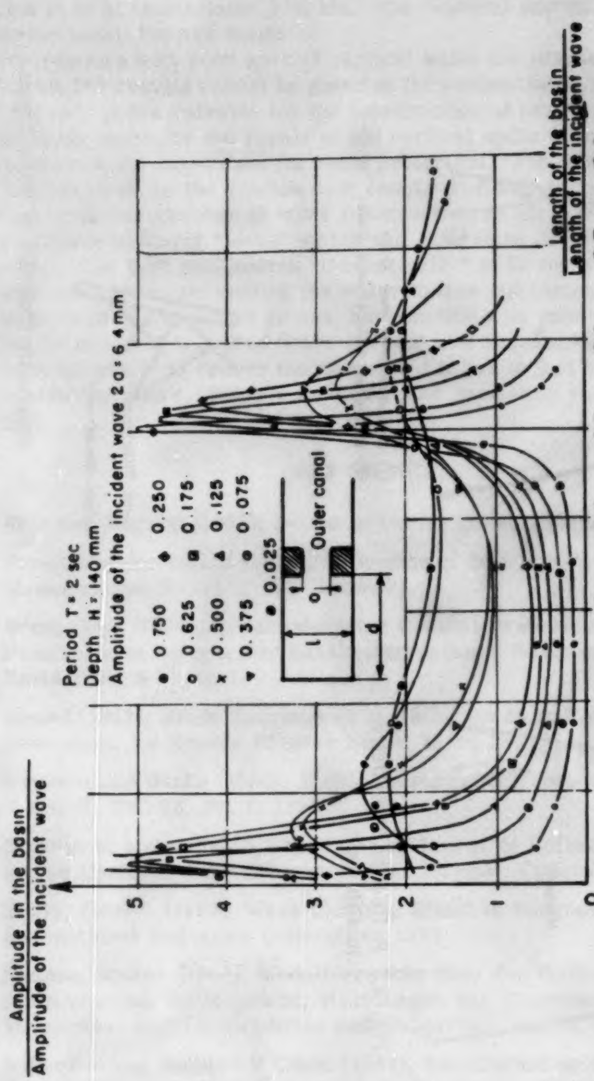

Fig. 4. Wave Agitation in a Basin Conditioned by an Opening, Restricted by Two Jetty Heads and by the Relative Length of the Basin (Reference No. 12).

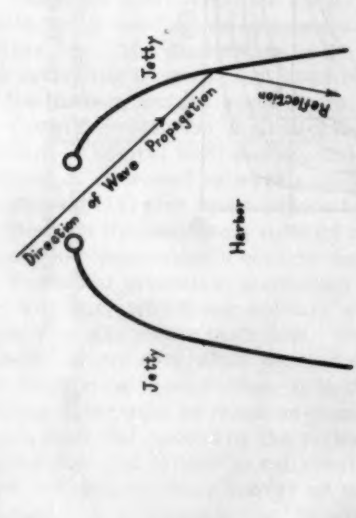

Fig. 5. Reflection from Jetty in a Harbor.

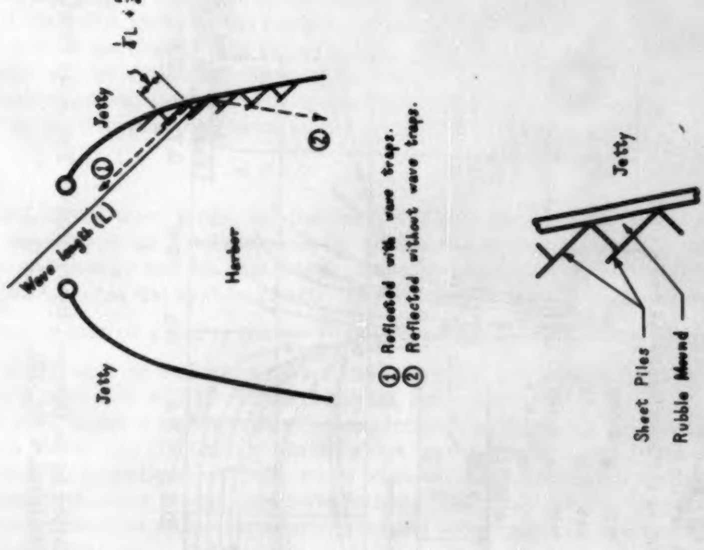

Fig. 6. Schematic Diagram of Wave Traps.

necessary therefore to protect the area of the beach at the foot of the vertical wall. Because the surf is a solitary wave, it is unimportant that it be reflected in two parts, spaced one half wave length. Less distance will be sufficient. Sometimes nature arranges itself in that way. Fig. 8 shows outcropping of coquina rock at Marineland, Florida. The "natural sea wall" consists of numerous small bays or tongues.

Experiments with such special vertical walls are planned at the University of Florida but results cannot be given at the present time. The experiments may not only prove valuable for the construction of new-type vertical walls but probably more for the repair of old vertical walls which will turn over if measures are not taken soon for their protection. Fig. 9 shows scour in front of a vertical wall on the Florida east coast. The wall is turning over.

Fig. 10 shows sketches of some construction which are under investigation. Some of these are only "ideas" which will have to be developed to fit practical purposes. The best wall seems "theoretically" to be the wall which entirely absorbs the uprush, permitting the water to flow out through pipelines which at the same time can act as groins, but practice will involve difficulties.

Fig. 11 shows a so-called 'wave screen' in a Dutch dike. The function of the wave screen is to reduce the uprush but it has in fact an effect similar to that mentioned above. Wooden wave screens have been very popular in England.

REFERENCES

1. Apte and Marcou (1955), Seiche in Ports, Coastal Engineering No. 5.
2. Beach Erosion Board (1952), Reflection of Solitary Waves, Technical Memorandum No. 11 (J. M. Caldwell).
3. Bruun, Per (1954), Breakwaters for Coastal Protection, Hydraulic Principles in Design, Permanent International Navigation Congress, Rome 1953, S II, Q. 1.
4. Biesel (1952), Etude Teórique de la Réflecion de la Houle sur certain Obstacles. La Houille Blanche No. A, Mars 1952.
5. Einstein and Banks (1950), Fluid Resistance of Composite Roughness, T.A.G.U. Vol. 31, No. 4, 1950.
6. Greslau et Mahe (1955), Etude du Coefficient de Reflexion d'une Houle sur un Obstacle Constitué par un Plan Incliné. Coastal Engineering No. 5.
7. Healy, John J. (1953), Wave Damping Effect of Beaches, Minnesota International Hydraulic Convention, 1953.
8. Hensen, Walter (1954), Modellversuche über den Wallenauflauf an Seedeichen im Wattengebiet, Mitteilingen der Hannoverschen Versuchsanstatt für Grundbau und Wasserbau, Heft 5, 1954.
9. Iribarren and Nogales Y Olano (1949), Penetration de l'Agitation dans les Ports; Moyens de la Prévoir et de la Combattre, Essais de Laboratoire - XVIIe Congres International de Navigation, Lisbonne 1949, S11, C. 4.
10. Johnson, Fuchs and Morison (1951), The Damping Action of Submerged Breakwaters, T.A.G.U., Vol. 32, No. 5, October 1951.

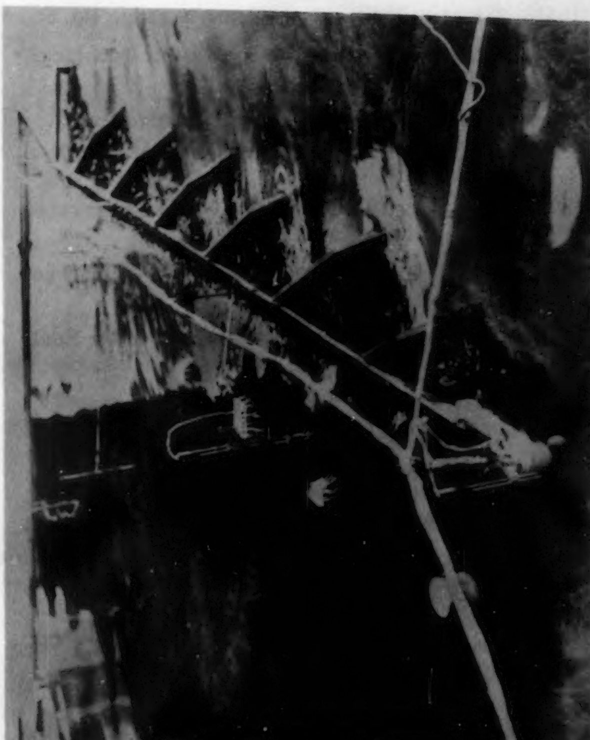

Fig. 7. Model Experiments with Wave Traps. Hydraulic Laboratories Technical University of Denmark.

Fig. 8. Natural Sea Wall Resulting from Outcropping of Coquina Rock, Marineland, Florida.

Fig. 11. Wave Screen Placed in a Sloping Wall, Holland.

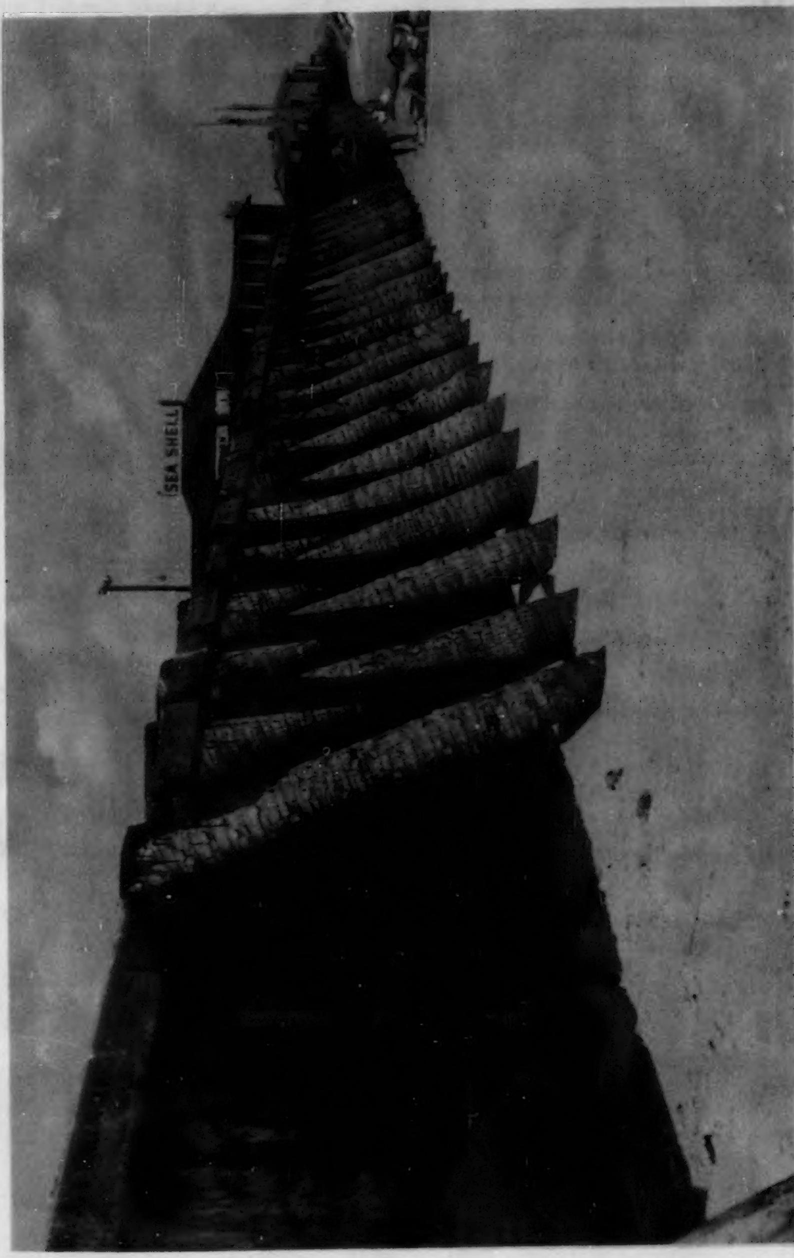

Fig. 9. Scour in Front of a Vertical Wall which has to be Supported. Jacksonville Beach, Florida.

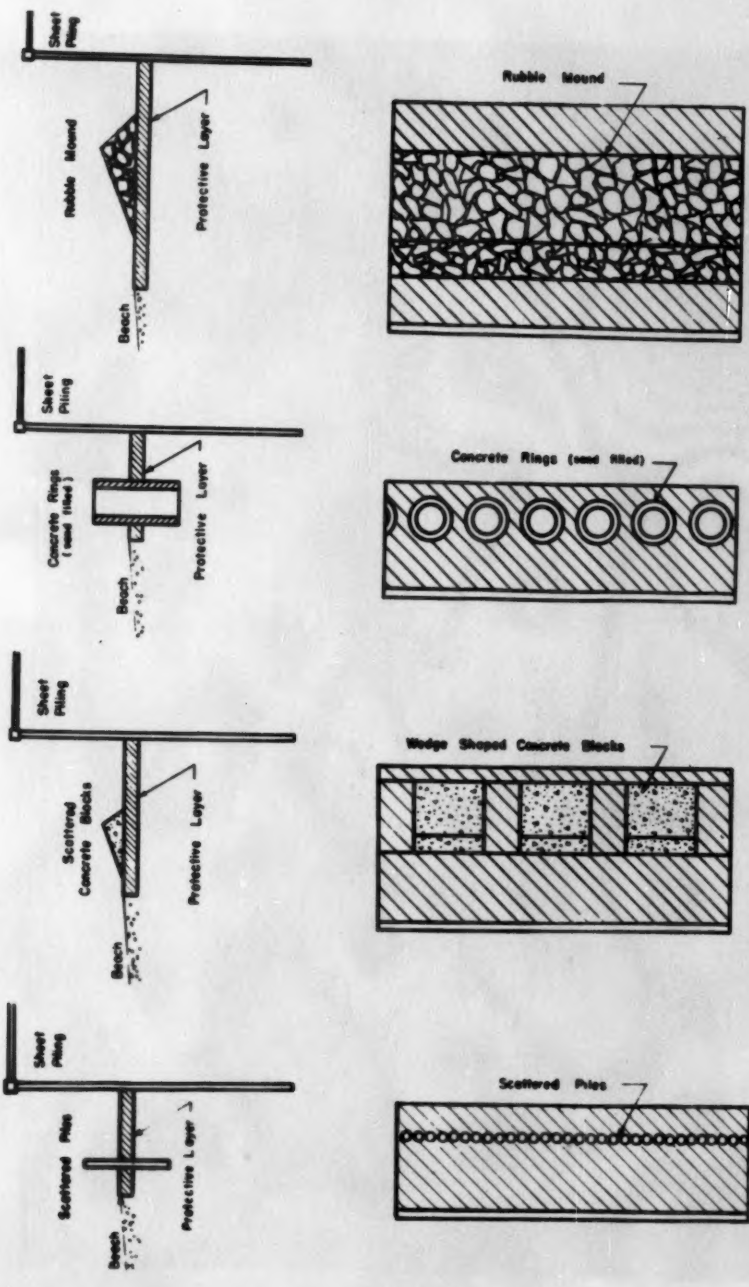

Fig. 10. Dividing Reflection at a Sea Wall, Shown Schematically.

11. Johnson, J. W., (1953), Engineering Aspects of Diffraction and Refraction, Transactions American Society of Civil Engineers, Paper No. 2556, 1953.

12. de Méhauté, B. (1955), Two-dimensional Seiche in a Basin Subjected to Incident Waves, Coastal Engineering No. 5.

13. Lundgren, H. (1950), Reflection from Vertical Permeable Walls. Hydraulic Laboratories, Copenhagen (unpublished).

14. Manohav, Modhav (1955), Mechanics of Bottom Sediment Movement due to Wave Action, Institute of Engineering Research, Berkeley, California, Series 72, Issue 2.

15. Miche, M. (1951), Le Pouvoir Réflechissant des Ouvrages Maritimes Exprés a l'Action de la Houle, Annales des Ponts et Chaussées, 1951.

16. Miche, M. (1944), Mouvement Ondulatoires de la Mer en Profondeur Constante ou Decroissante, Annales des Ponts et Chaussées, 1944.

17. Putnam, J. A. (1949), Loss of Wave Energy due to Percolation in a Permeable Sea Bottom, T.A.G.U. Vol. 30, No. 3, June 1949.

18. Schlichting (1936), Ingenieur-Archiv. J. Band, I. Heft.

19. Schoemaker and Thijsse (1949), Investigation of the Reflection of Waves, International Association for Hydraulic Structures Research, Grenoble 1949.

20. Schoklitsch, A. (1950), Handbuch Des Wasserbaues I, Vienna.

21. The Dock and Harbour Authority (1952), Pneumatic Breakwaters, May 1952.

22. The Dock and Harbour Authority (1952), Pneumatic Breakwater at Dover, December 1952.

23. Valembois, J. (1954), Etude de l'Action d'Ouvrages Resonants sur la Propagation de la Houle, Proceedings Minnesota International Hydraulic Convention, 1953.

24. Vaoni and Carr (1952), Harbour Surging, The Dock and Harbour Authority, June 1952.

25. U. S. Navy Department (1947), Wind Waves at Sea. Breakers and Surf. H. O. Pub. No. 602.

10. [illegible] (1953), [illegible] Aspects of [illegible] Transportation, American Society of Civil Engineers, Paper No. [illegible], 1953.

11. [illegible]

12. [illegible]

14. [illegible]

15. [illegible]

16. [illegible]

17. [illegible]

18. [illegible]

19. [illegible]

20. Schoklitsch, A. (1950), Handbuch des Wasserbaues, Vienna.

21. The Dock and Harbour Authority (1953), [illegible], May 1953.

22. The Dock and Harbour Authority (1951), [illegible], December 1951.

23. [illegible]

24. [illegible] (1952), [illegible], The Dock and Harbour Authority, June 1952.

25. U.S. [illegible] Department [illegible]

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DISCUSSION
(Proc. Paper 913)

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Discussion of
"DEVELOPMENT OF THE DELAWARE RIVER FOR COMMERCE"

by B. B. Talley
(Proc. Paper 503)

CLARENCE RENSHAW.[1,2]—The answer to Colonel Hall's question "Has there ever been a depth of 40 feet at mean low tide over the entire stretch from Philadelphia to New Castle?" is no. The Delaware River channel was deepened from 35 to 40 feet over most of its length during the period 1940-1945 primarily as a defense measure. However, the war ended before the channel was completed and the existing traffic after the war did not warrant maintenance to 40 feet. The channel was maintained to 37 feet from 1946 to 1953. Because of the use of deep-draft tankers and ore carriers with drafts up to 37 feet, local interests requested in February 1954 that the channel be restored to its project depth by January 1956. This work is under way.

The elimination of the discharge of culm from the Schuylkill River and improved dredging methods will aid in obtaining and maintaining project depth in the Delaware. Dredging methods employed in the past permitted a great deal of fine material to re-enter the channel which resulted in shoal soundings on the sonic depth-finder. The "sump rehandler" method of dredging, which has been employed since January 1955, places all the dredged material in disposal areas on shore, thereby eliminating the return of fines to the channel.

It is estimated that the proposed 40-foot channel from Philadelphia to Trenton will require the removal of 1,641,000 cu. yds. annually to maintain project depth. It is not expected that maintenance of this section of the channel will be difficult.

1. This is the closing discussion prepared at the author's request in his absence.
2. Colonel, Corps of Engrs., U. S. Dept. of the Army, Div. Engr., North Atlantic Division, New York, N. Y.

Discussion of
"DEVELOPMENT OF THE DELAWARE RIVER FOR COMMERCE"

by E. H. Thiele
(Proc. Paper 505)

CLARENCE RENSHAW,[1,2]—The answer to Colonel Hall's question "Has there ever been a depth of 40 feet at mean low tide over the entire stretch from Philadelphia to New Castle?" is no. The Delaware River channel was dredged from 35 to 40 feet over most of its length during the period 1942-1944 primarily as a wartime measure. However, the war ended before the channel was completed and the shipping traffic after the war did not warrant maintenance to 40 feet. The channel was maintained at 37 feet during 1946 to 1952. Because of the use of deep-draft tankers and ore carriers with drafts up to 37 feet, local interests requested in February 1954 that the channel be restored to its project depth of January 1956. This work is under way.

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1. This is the closing discussion submitted at the author's request by his successor.
2. Colonel, Corps of Engrs., U. S. Dept. of the Army, Div. Engr., North Atlantic Division, New York, N. Y.

Discussion of
"GRAPHIC DESIGN OF ALLUVIAL CHANNELS"

by Ning Chien
(Proc. Paper 611)

NING CHIEN,[1] A.M. ASCE.—Professor Lane stated correctly that research on the determination of alluvial channel width is urgently needed, and his outline of the factors involved in the problem should prove to be helpful in planning such a research program. The writer wishes to add another factor—the secondary current, which is believed to have an important effect on both the channel width and the meander pattern.

The writer has some difficulties in following the arguments presented by Mr. Blench. If the use of flume data to river conditions represents a "bold extrapolation," then, considering the differences between irrigation canals and rivers like the Missouri or Mississippi, surely the "extrapolation" of the observations from canals to rivers is just as "bold." The design diagrams presented by the writer are constructed according to the Einstein method which is derived from measurements covering a wide range of conditions, as follows:

Slope from 0.015 to 2.0 percent
Particle size from 0.8 to 30 mm.
Water depth from 0.03 to 4 ft.
Discharge from 0.0215 to 21.5 cfs/ft.
Sp. gr. of sediment from 1.05 to 4.2
Particle shape from near spherical and flat to rectangular, and has been checked in various rivers.

Recent extension of the Theory(A) also covered sediment mixtures with sizes varying from 4 mm. down to 0.005 mm., and with sorting coefficient as high as 2.5, which is higher than most of the natural river sediment.(B) The use of light-weight material actually extends the range of flow conditions even more than that listed above, according to any suitable model laws (including Blench's). In fact, the writer has conducted a special set of flume experiments for the Corps of Engineers by using light-weight material (sp. gr. 1.05) with the same gradation as Missouri River sand, and by reproducing the Missouri River flows from low stages to floods according to the model laws. The results thus obtained substantially follow the theories advanced by Professor Einstein. And yet Mr. Blench claimed that all these data are either "trifle" in range, or "unnatural" in material, or "poor" in measurements. On the other hand, he seems never to be bothered by the fact that the regime theory was developed in the Indian plain where the bed material size varies only from 0.05 to 0.38 mm.(C)

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1. Associate Prof. of Hydraulics, Tsing Hwa Univ., Peiping, China; formerly Assistant Research Engineer, Institute of Engineering Research, University of California.

The writer has assumed uniform material and has ignored the bank friction in constructing the design curves, such that these curves will be sufficiently general to be used to illustrate their various possible use in aiding the planning of river control works. It has never been the writer's intention that these curves are the curves to be used by every river engineer. In fact, he has specifically pointed out in his paper that one should take all the local conditions, including the bank friction and the gradation of bed material, into consideration and construct more reliable curves for the individual case using the same basic methods.

A much more serious question raised by Mr. Blench's criticism is the following: Can we rely on flume data to predict river conditions? The answer to this question depends on the types of information to be sought for, and also on the way the flume experiments are conducted. In the particular problem of sediment transportation, the basic mechanics of transport is set upon the basis of both actual observations and physical reasonings. Such a description on the basis of the forces which move the sediment actually studies the way it moves, the factors which control its rate of movement and the mutual effect between the liquid and solid phase of the mixture, and equally applies to flumes and to rivers, since most important dimensionless parameters have equal values in both scales. The transformation from theoretical considerations into practical engineering tools, however, involves the determination of certain constants of functions. This is accomplished by conducting flume studies, and the limitations imposed on the flumes naturally are also inherent in the results obtained from the flumes. The limitation on the range of flow conditions available in laboratories can partly be overcome by using lightweight material as sediment which moves more easily in the flow. On the other hand, it is obvious that we cannot study meander patterns in a narrow channel. It has also been found that the bar resistance does not develop to its full effect in a narrow channel. Such deficiencies in flume data must be supplemented by using actual river measurements. Thus, Einstein and Barbarossa(4) derived the bar resistance effect from the river data, and, for the same reason, cannot be used to narrow irrigation canals without certain modifications.

On the other hand, the flume flow, because of its confinement by two sidewalls, is affected to a higher extent by bank friction than the natural river flows. Einstein has devised a method(D) to remove this bank effect from the over-all flow resistance. This method, incorrect as it may appear to Mr. Blench, has its physical background.(3,E) It has also been verified repeatedly by independent measurements,(F,G,H) and is now generally accepted as a standard procedure in analyzing flume data. As the theory on sediment transport thus developed is not directly affected by the bank friction, its range of applicability is again greatly extended. Mr. Blench, however, preferred the use of "original" data, which include the effect of bank friction. He has also stated(I) that in the Indian plains where the regime theory originated, the banks behave hydraulically smooth. The question immediately arises as to whether the same theory which includes the effect of friction of smooth sidewalls, can be applied to channels with rough banks. Again, Mr. Blench has never made this point clear in his writings.

Mr. Blench, based on his experience with one single case, claimed that the Einstein method is wrong, both "functionally" and "factually." There are a number of questions to be considered before this outright accusation can stand up against further scrutiny. Is the material in this case uniform in size or a

distinct mixture? What is the actual sediment supply? Is it 0.1, 1, or 10 tons/day/ft? Has it been measured? If so, does it include also the part of the bed-material load which is in suspension? What is the slope of the channel? Is the channel really alluvial? On the other hand, Mr. Blench evidently ignored many field measurements which essentially prove the reliability of the Einstein method in determining sediment transport rate of natural river channels. In this respect the writer is in full agreement with Professor Lane[J] that the greatest need in the sediment engineering field today is extensive field measurement of the sediment rate transported in channels under a wide range of conditions, against which the various theories may be thoroughly checked. Before this is done, the Einstein method, because of its physical soundness and its coverage of a wide range of conditions, is likely to be the best tool available to the field engineers.

REFERENCES

A. "Transport of Sediment Mixtures with Large Ranges of Grain Sizes," by H. A. Einstein and Ning Chien, M.R.D. Sediment Series No. 2, Corps of Engineers, Missouri River Division, 1953.

B. "Discussion of the Paper 'Mechanics of Streams with Movable Beds of Fine Sand,' by N. H. Brooks, Proc. Separate No. 668," by H. A. Einstein and Ning Chien (not yet published).

C. "A Concept of Lacey's Regime Theory," by Ning Chien, Proc. Separate No. 620, ASCE, Feb. 1955.

D. "Formulas for the Transportation of Bed Load," by H. A. Einstein, Trans. ASCE, Vol. 107, 1942, pp. 561-573, first published as "Der Hydraulische oder Profil-Radius," by H. A. Einstein, Schweizerische Bauzeitung, Vol. 103, No. 8, Feb. 24, 1934.

E. "Der Hydraulische oder Profil-Radius," by H. A. Einstein, Schweizerische Bauzeitung, Vol. 103, No. 8, Feb. 1934.

F. "Mean Roughness Coefficient in Open Channels with Different Roughness of Bed and Side Walls," by A. M. Yassin, Mitteilungen Aus der Versuchsanstalt für Wasserban und Erdban, Zurich, Nr. 27, 1953.

G. "Experiments on a Rectangular Flume with Sides and Bed of Different Roughness," by J. A. Kerr (unpublished), 1953.

H. "Experiments in a Rectangular Flume with Sides of Glass and a Bed of Two Superimposed Roughnesses," by J. A. Kerr (unpublished), 1953.

I. "Hydraulics of Sediment-Bearing Canals and Rivers," by T. Blench, Evans Industries Limited, Vancouver, B.C., Canada, 1951.

J. "Discussion of the paper 'A Concept of Lacey's Regime Theory' by Ning Chien," by E. W. Lane, Proc. Separate No. 620, 1955.

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REFERENCES

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Discussion of
"RIVER SURVEYS IN UNMAPPED TERRITORY"

by Gerard H. Matthes
(Proc. Paper 612)

GERARD H. MATTHES,[1] HON. M. ASCE.—The discussions have contributed valuable points derived from actual experience. They have been greatly appreciated by the writer, also have impressed upon him the need for clarifying the limitations besetting the relations between slope, depth and rolling diameter shown in Figure 1. These relations are strictly applicable only to rivers whose bed load has suffered no loss by interception by passing through lakes or by flow diversions of any kind. Furthermore, they are applicable only to rivers whose bank-full flow is carried within a channel free from dams, islands, split-channel conditions and from flow diversions, whether into floodways, head races of power developments or irrigation canals. These all rob the main channel of bed load. The rather severe limitations here cited were kept in mind by the writer whenever he made searches for bed-load specimens. In the course of his endeavors he discovered, first in the Mississippi River, and later again in the Sacramento River (before any high dams were built on the latter) that a low swale across the narrow neck of a bend afforded unusual opportunities for finding coarse bed-load material. The reason is, water diverted through such a swale during a high flood stage consists mainly of bottom water plus bed-load. The significance of bed-load diversions of this kind was first made clear by laboratory studies conducted by H. Thoma at Munich, 1919, confirmed by Th. Rehbock(1) 1922 in the hydraulic laboratory at Karlsruhe, and carried out in considerable detail in 1932 at the U. S. Waterways Experiment Station, Vicksburg, Miss. at the writer's request. These experiments demonstrated that a fifty per cent flow diversion may carry with it as high as 80% of the river's bed-load. These experiments revealed furthermore that at a sharp river bend, a 10% flow diversion made along the convex bank entrained up to 90% of the bed-load. In all these cases it was found that the swift top water, owing to its momentum, suffered no appreciable diversion, the water diverted being chiefly bottom water carrying the bulk of the bed-load. The point raised by Mr. Kuiper to the effect that the largest rolling diameter of gravel found by him in the lower reaches of the Saskatchewan River did not check with the relations shown in Figure 1 is quite understandable when account is taken of the fact that many of the Saskatchewan's tributaries are noted for the large lakes which they traverse. These tributaries are thus prevented from supplying coarse bed-load material to the parent stream. This consideration affects particularly the lower reaches of the river. Mr. Kuiper's methods of estimating mean velocity, mean depth and discharge from a boat floating down the axis of the current are ingenious, thoroughly practical and emphasize the usefulness of aerial photographs in exploratory river surveys.

Professor Posey's method of establishing by means of a flash the time intervals during photographic recording of floats in model experiments, is

1. Cons. Hydr. Engr., New York, N. Y.

simple and worthy of adoption elsewhere. Professor Posey questions the occurrence of super-elevation of a stream's water surface along the axis of flood flow, except where the channel's cross-section changes, as at bends. The writer has observed noticeable super-elevation in straight reaches of many streams. During a sudden rise in a mountain stream in California, witnessed by him, the upward thrust along the axis of swiftest flow not only raised the central water surface appreciably, but lifted small boulders to its surface at intervals. A flood in Cherry Creek in Denver, where its channel runs straight, confined between concrete walls, afforded him an opportunity to note the central rise of the water surface, which was sufficient to hide from view the top of the concrete wall along the side opposite. Some conception of what takes place within the cross section of a torrential stream in flood is made apparent by watching a mud flow descend from a steep ravine into the plain—a common occurrence down the front of the Wahsatch Mountains in Utah. Not only is the center of a mud flow appreciably higher than its edges, but large rock fragments are forced upward within it and ride on its bulging surface for some distance, before sinking out of sight. The nature of this phenomenon needs study, but presents many practical difficulties for reproduction in a laboratory.

The contribution made by Mr. Nevin, Hydraulic Engineer, New Zealand's Soil Conservation and Rivers Control Council, is particularly welcome, as it sums up in a measure the concensus of 15 years of experience of a highly trained staff of river engineers, whose exclusive task concerns the control of some of the world's most unusual rivers. Their bed loads consist preponderantly of shingle (slab rock) hence lack unfortunately the rolling diameter factor needed for plotting in Figure 1. Yet these rivers, upon debouching into the coastal plains, build as well-shaped meanders as do rivers transporting sand and gravel. Mr. Nevins' reference to the formula for minimum radii of stable bends in alluvium, recommended by the late A. P. Grant, is worthy of note by American river engineers. Grant's works stamp him as one of the ablest river engineers of his time. The formula has its practical applications in connection with river surveys in unmapped territory as pointed out by Mr. Nevins.

REFERENCE

1. Hydraulic Laboratory Practice," by John R. Freeman, pub. by Amer. Soc. of Mechanical Engineers, New York, 1929, pp. 179-180.

Discussion of
"THE DESIGN OF PIERS, JETTIES AND DOLPHINS"

By David A. Hopkins
(Proc. Paper 727)

ZUSSE LEVINTON,[1] M. ASCE.—The author is to be congratulated on making a simple and concise statement of design methods relating to ship impact on piers. The two examples of piers on flexible piling show how the impact energy may be absorbed by the deflection of the structure. However, it would be erroneous to conclude that, in general, flexibility of the structure itself is preferable to flexibility provided by resilient fendering. A high degree of flexibility in the structure generally goes together with weakness in the structure. This point is illustrated in the author's statement that when "the comparative weakness of the pier was appreciated, it was decided that some stiffening of the ends was necessary." In the same example, "the ends of the pier are protected by 40-foot diameter sheetpile caisson dolphins, which are located at each end of the pier." Having thus protected the ends by stiff structures, the middle could be made flexible to allow a computed deflection of 5.8", which was possibly expedient in this case.

In general, it is not desirable to have a pier bounce as much as six inches from ship impact, a condition similar to earthquake shock. For such deflections, special flexible connections may be required for pipe lines and utilities on the pier, and special provisions in the design of superstructures, such as cranes, conveyor towers or hoppers. There is also a matter of inconvenience and even danger to the personnel.

The advantage of lesser wear and tear of the rubbing face, when the structure is flexible, is correctly pointed out by the author. The same advantage will, of course, ensue when the rubbing face is backed up by flexible springs attached to a stiffer pier structure.

In the second case cited by the author, the pier is wide, with a large number of piles in each bent. The computed 3" deflection is rather small and would be even smaller if the soil were stiffer, and the point of virtual fixity of the piles much higher. Under such conditions the structure would require resilient fendering.

In the writer's practice, there were numerous cases where fewer and heavier piles were preferred to larger numbers of lighter piles. This resulted in stronger and stiffer structures, where the necessary flexibility was provided by spring fendering. The economy in the structure was more than enough to cover the cost of the springs.

Higher velocities of approach and larger ships require greater deflections for absorption of the energy. Deflections of 12" to 18" are quite easily obtainable by various types of steel springs. To incorporate such flexibility in the structure itself is impractical.

The writer wishes to point out that in the author's examples of 14,000 ton and 25,000 ton ships, the term "displacement tonnage" should be used rather than "deadweight tonnage."

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1. Chf. Str. Engr., Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

Discussion of

THE DESIGN OF [illegible]

[illegible]

[illegible][1], M. ASCE.—The author [illegible] the [illegible] design [illegible] a plate [illegible]. The [illegible] may be [illegible] to the [illegible]. [illegible] A high degree of [illegible]. This point is illustrated [illegible] of the plate [illegible] the [illegible] of the [illegible]. In the [illegible] the plate [illegible] of each end of the [illegible] the [illegible] could be [illegible] to [illegible] a [illegible] which was [illegible].

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Higher [illegible] and larger [illegible] deflections for [illegible]. Deflections of 12" to 15" are quite [illegible] of steel [illegible]. To [illegible] the structure [illegible].

The writer wishes to point out that in the author's [illegible] the term "[illegible]" should be used rather than "[illegible]."

1 Chf. Str. Engr., [illegible], New York, N. Y.

Discussion of "CONTROL OF ARROYO FLOODS AT ALBUQUERQUE, NEW MEXICO"

by Rufus H. Carter, Jr.
(Proc. Paper 801)

GEARY M. ALLEN, Jr.,[1] J.M. ASCE.—The author is to be congratulated for his succinct and straightforward presentation of a highly complex flood problem which is at present threatening the future and growth of a large portion of the City of Albuquerque. During 1955 there were recorded at least nine damaging floods from the arroyo watersheds east of the City. The damages were mainly sustained by a developed area in the northern valley section of the city. Many of the hard hit areas have been depopulated because of the persistence of flooding during 1955.

The Albuquerque District, Corps of Engineers, has the responsibility of construction of the Albuquerque Arroyos diversions project which when completed and augmented by the program of the local Sandia Conservancy District will provide complete flood protection for the City of Albuquerque from the devastating effects of the floods originating on the arroyos east of the city. The project is now in final design stage.

The writer's particular interest lies in the hydraulic design of the diversion channels described by the author. The diversion plan presented to the Congress by the Secretary of the Army, and upon which project authorization is based, contemplated that the main diversion channels would be designed to convey the flood discharges at depths greater than the critical depth (subcritical velocities). Of necessity therefore, large ponding areas at each arroyo junction would be required, not only to obtain sufficient hydrostatic head to force the flow into the diversion channel but to permit a certain amount of sediment deposition at the entrance to the diversion channel. Although subcritical velocities would be obtained in the diversion channel, the design flood velocities would range from 12 feet per second to 23 feet per second; therefore, concrete lining is required for protection against scour. Since concrete lining is required for the diversion channel regardless of the fact that the state of flow is subcritical, the Albuquerque District, Corps of Engineers, is currently investigating the feasibility of designing the diversion channels to convey flood waters at supercritical velocities. The design flood velocities would range from 30 feet per second to 35 feet per second. Since supercritical flow requires a smaller flow area for its conveyance, some savings in concrete and reinforcing steel will result if the plan proves feasible, In addition, the large ponding areas at the arroyo junctions with their attendant sedimentation problems would be eliminated, and crossing structures would be shortened. Flow depths would be designed at or less than 80% of the critical depth in order to assure stable flow.

Naturally a channel conveying flow at velocities exceeding 30 feet per second must be designed more carefully and is less flexible in alignment than

1. Civ. Engr., Corps of Engrs., Albuquerque Dist., Albuquerque, N. Mex.

a channel designed for 15 feet per second. An example of this is the treatment of horizontal curves. A velocity of 32 feet per second in the 15 foot bottom width channel and a horizontal curve radius of 500 feet would develop a difference of about 4.5 feet between the water surface at the outside and inside of the curve. Such distortion of depths and velocity distribution sets up wave disturbances which persist far down into the downstream tangent. Therefore, considerable care must be taken in selecting an alignment within the governing alignment restrictions which permits the use of large radii horizontal curves. Also, additional freeboard must be allowed to compensate for these wave disturbances since super-elevation of the bottom of a trapezoidal shaped channel to compensate for the centrifugal effect is not practical. Although the trapezoidal shape selected for the diversion channel is admittedly not the most efficient hydraulic shape for conducting supercritical flow around horizontal curves, the selection of the trapezoidal section in lieu of rectangular was primarily an economic consideration. Although increased free board is necessary to contain the flow a considerable economic advantage remains in favor of the trapezoidal section.

Another hydraulic problem of major significance attendant with the supercritical channel is posed at the junction of each major arroyo and the main channel. There will be at least six major junctions in the North Diversion channel alone. Because of the importance to the success of the project of maintaining the supercritical state of flow through the junction section it is necessary that the arroyo discharge be introduced into the main channel in a manner which will prevent unusual surging and wave disturbance and with as little loss of energy as possible. At present the Albuquerque District is studying the feasibility of placing the arroyo entrances on a spiral transition curve alignment, which would become tangent to the main channel. The spiral transition entrance has been demonstrated to be very successful at the confluence of two rectangular sections by the Los Angeles District, Corps of Engineers. The water surface profile through the junction reach can be approximated by a momentum analysis[2] developed by the Los Angeles District and verified to be workable by model study.

2. Hydraulic Model Study Los Angeles River Improvement, Whitsell Avenue to Tajunga Wash , L.A. County, Calif., Los Angeles District, Corps of Engineers, July 1949.

Discussion of
"FLOOD CONTROL IN THE MIDDLE MISSISSIPPI"

by Walter F. Lawlor
(Proc. Paper 803)

E. KUIPER.[1]—While reading Mr. Lawlor's interesting paper on the Mississippi River, the writer was somewhat puzzled by the following statement. "Investigations indicated that relatively higher stages can be expected when a large percentage of the flow comes from the Missouri River, which carries a large bed load of material; during summer floods; and also when the rate of rise of the flood wave is relatively slow."

Was it the intention of the author to indicate that a higher stage was caused by a higher river bed, which in turn was caused by the above mentioned factors, or was it meant that the river bed was constant and that the total depth of flow was increased through an increase in channel resistance? In the latter case, the statement becomes most interesting and the writer would greatly appreciate a description and explanation of these observed phenomena, since so little is known and published about the variation of hydraulic roughness as a function of sediment transport and other variables.

1. Senior Hyd. Engr., P.F.R.A., Dept. of Agriculture, Canada.

DIVISION ACTIVITIES

WATERWAYS DIVISION

Proceedings of the American Society of Civil Engineers

NEWSLETTER

March, 1956

There has been a gratifying increase in the interest in the Waterways Division during the past several years. We feel that this is due to a large degree to the efforts of the members of the Executive Committee and particularly to the past chairmen who have furnished the inspiration.

We are, of course, interested in increasing the enrollment in the Waterways Division. Many members of the Society who are interested in inland waterways, harbors, ports, or coastal engineering and whose daily activities are in one or more of these fields are not registered in this Division. Some are registered in other divisions. Others have not taken the trouble to register in any division. We wish to urge all such members of the Society to register in the Waterways Division.

As you will learn elsewhere in this Newsletter, the Executive Committee has proposed a change in the designation of the Division. We believe that the new designation will give a better idea of the scope and interest.

The value of a technical division is proportional to the participation of its members in its activities and production. We believe that there are many in the Society who could and would like to contribute to our division in the production of papers or work on committees but have not been contacted. If you would like to participate in this important work a letter to the secretary, a member of the executive committee, or to a technical committee chairman would be most welcome.

Charles M. Wellons
Chairman
Executive Committee
Waterways Division

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MEETING OF EXECUTIVE COMMITTEE

October 26, 1955

The annual meeting of the Division Executive Committee was held in New York during the ASCE Convention.

Members of Executive Committee

Clarence C. Burger, Jr. (Chairman, 1954-55), Chief, Operations Division, Office of Chief of Engineers, Washington

Charles M. Wellons (Vice-Chairman, 1954-55; Chairman, 1955-56), Consulting Engineer, Pittsburgh

Lewis C. Coxe (Vice-Chairman, 1955-56), Capt. CEC, USN, Public Works Officer, U. S. Naval Academy, Annapolis

Carl B. Jansen, President, Dravo Corporation, Pittsburgh

Rufus W. Putnam, Executive Officer, State Lands Commission, Los Angeles

Roger H. Gilman (Secretary), Director of Port Development, Port of New York Authority, New York

Contact Member, Board of Direction, ASCE

Carey H. Brown, Manager, Engineering and Manufacturing Services, Eastman Kodak Co., Rochester

Chairman Burger distributed his Annual Report for 1954-1955, which reviewed the activities of the Division for that period. He stressed the excellent work which had been done, particularly by the Committees on Publications and Session Programs, in carrying out their responsibilities throughout the entire year. The Chairman also remarked on the value of attendance at the Technical Procedures Conference by Officers of the Division, especially for the Vice-Chairman, who will assume responsibility for the affairs of the Division in the following year.

Secretary Gilman reviewed briefly a few Division matters. As of July 1, Division expenditures were within the budget for 1954-1955, although most expenses were for administrative and Executive Committee matters and practically none for Technical Committees. He noted that the recommended budget for 1955-1956 again totals $2100, and expressed a hope that the Technical Committees would make full use of their funds for meetings and other purposes. Mr. Burger emphasized this point, remarking that the ASCE officers at the Technical Procedures Conference had advised the Divisions that they should proceed with worthwhile and authorized projects and that the approved budget should not be regarded as an absolute top limit if additional expenditures were found necessary.

The Executive Committee considered the desirability of closer liaison or possible affiliation by the Waterways Division with other organizations such as the American Section, Permanent International Association of Navigation Congresses (PIANC) and the American Association of Port Authorities

(AAPA).* Mr. Burger will continue to explore this idea with PIANC and Messrs. Putnam and Gilman will do the same with the AAPA. It was also agreed that the Committee on Session Programs would extend an invitation to other organizations whose members might be interested in attending the Waterways Division sessions at ASCE meetings where and when appropriate.

The Committee chairmen reviewed their activities during the past year and discussed their programs for the future.

Capt. Coxe then expressed the view that the name of the Waterways Division was not broad enough to indicate its wide scope and purpose in the field of harbor and port development. The responsibility is clearly evident in the defined purpose of the Division. There was unanimous agreement, therefore, that the Executive Committee should recommend to the Society that the name be revised to Waterways and Harbors Division. We feel certain that this will be helpful in informing ASCE members that the Division is not confined in its scope to inland waterways matters but includes the important field of ports and harbors, which has come into great prominence, particularly in recent years.

The subject of research was then discussed. The Society is apparently anxious to develop a list of major problems in all phases of civil engineering which could be the subject of research. Topics were submitted by Division members, in response to a recent Newsletter request, but it was felt desirable to recommend establishment of a Committee on Research which would be responsible for initiating, organizing, sponsoring, and coordinating research in the field of waterways and harbors. This Committee of the Division would then cooperate with the Committee on Research of the Society. Accordingly, the Executive Committee recommended the formation of a Committee on Research in the Waterways Division. The purpose of the Committee would be:

> "To initiate, organize, sponsor and coordinate research in the field of waterways and harbors and to further the advancement of waterways and harbors through the coordination of endeavor with divisions having related interests on specific subjects."

The Secretary then reviewed the recent revision in the by-laws as to membership on the Executive Committee. The members unanimously agreed that Mr. Burger should remain on the Executive Committee until the end of his term in 1956, so that we can have the benefit of his counsel and advice. Mr. Gilman, who was designated earlier in the year as the new member of the Executive Committee, was also re-elected as Secretary until October 1956.

For the year 1955-56, Charles M. Wellons was elected as Chairman and Capt. Lewis C. Coxe was elected as Vice-Chairman of the Waterways Division.

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* At the November Annual Convention of the American Association of Port Authorities in Houston, the AAPA Committee on Port Construction and Development discussed the possibility of developing, in cooperation with the ASCE Waterways Division Committee on Ports and Harbors, a "Manual of Port Layout and Planning."

SUMMARY OF COMMITTEE ACTIVITIES

Committee on Session Programs

The membership of the committee is as follows:

Lawrence B. Feagin (Chairman), Chief, Operations Division, Mississippi River Commission, Corps of Engineers, Vicksburg

Charles M. Wellons, Consulting Engineer, Pittsburgh

R. R. Shoemaker, Chief Harbor Engineer, Port of Long Beach, Long Beach

Walter F. Lawlor, Chief, Engineering Division, St. Louis District, Corps of Engineers, St. Louis

Robert J. Winters, Coordinator of Engineering, Port of New York Authority, New York

Donald H. McCoskey, Chief, Engineering Division, Southwestern Division, Corps of Engineers, Dallas

Gilbert M. Dorland, Colonel, Corps of Engineers, District Engineer, Nashville District, Nashville

The Committee arranged technical programs for three conventions during the last year, and plans for several conventions during the present year are well underway.

San Diego Convention, February 1955. Participation by the Waterways Division consisted of a technical session of three papers, and a second session sponsored jointly with the City Planning Division with four papers. The program was arranged by Mr. Shoemaker and presided over by Col. Putnam of the Division Executive Committee. Attendance was about 40-50.

St. Louis Convention, June 1955. Participation by the Waterways Division consisted of two technical sessions, and joint sponsorship of a luncheon. The morning technical session consisted of four papers. The luncheon, co-sponsored by the Waterways, Hydraulics and Soil Mechanics and Foundations Divisions, featured an address by Brig. Gen. John R. Hardin, President, Mississippi River Commission and Division Engineer, Lower Mississippi Valley Division, on the subject "The Problem of Control of Old River." The afternoon session, also co-sponsored by the Waterways, Hydraulics and Soil Mechanics and Foundations Divisions, consisted of a Symposium of four papers on "Old River Diversion Control." A field trip sponsored by the Division was made by about forty to the Chain of Rocks Locks and Canal project. The program was arranged by Mr. Feagin, while Mr. Lawlor handled the meeting details. The morning and afternoon sessions were presided over by Mr. Wellons, Vice-Chairman of the Division Executive Committee. Attendance averaged about 75-100 at these two sessions. The luncheon was presided over by Mr. Burger, Chairman of the Division Executive Committee.

New York Convention, October 1955. Participation by the Waterways Division consisted of an afternoon panel program on "Bridge Clearances," presided over by Mr. Burger, Chairman of the Division Executive Committee. This program was jointly sponsored by the Highway Division. There were two technical sessions of eight papers on Coastal Engineering subjects, planned by the Committee on Coastal Engineering under the Chairmanship of Mr. Eaton. These sessions were also presided over by Mr. Burger. Mr. Robert J. Winters is the New York member of the Committee on Session Programs.

Dallas Convention, February 1956. Mr. D. H. McCoskey, Chief, Engineering Division, Southwestern Division, Corps of Engineers, Dallas, Texas, arranged the program for the Dallas Convention. The program consisted of two technical sessions of four papers each.

Knoxville Convention, June 1956. Col. Gilbert M. Dorland, District Engineer, Nashville District, has been designated as the committee member to develop the program for the Knoxville Convention. It is proposed that the Waterways Division sponsor two technical sessions of about four papers each.

To facilitate longer range planning, it is proposed to ascertain from the Chairmen of the Technical Committees of the Waterways Division, their wishes regarding participation in future technical sessions, with the objective of encouraging and properly scheduling technical papers of special interest to the profession.

Committee on Publications

The membership of the committee is as follows:

Evan W. Vaughan (Chairman), Parsons, Brinckerhoff, Hall and MacDonald, New York

John M. Buckley, Consulting Engineer, Department of Marine and Aviation, City of New York, New York

Ellsworth I. Davis, Col., Office of Secretary of the Army, Washington

Jay V. Hall, Jr., Chief, Engineering Division, Beach Erosion Board, Corps of Engineers, Washington

Joseph M. Caldwell, Hydraulic Engineer, Beach Erosion Board, Corps of Engineers.

The principal activity of the Committee has been to maintain operational procedures for, and to review and recommend papers for publication as Proceedings—Separates and Transactions. Tentative procedures were established with the approval of the Executive Committee of the Waterways Division in May 1954. Since that date, some modifications in detail were found desirable and these have been made up as required. A meeting of the Committee was held during the October Convention.

By action of the Board of Direction of the Society, after October 1955 the Divisions will not be required to review and recommend Proceedings-Separates for publications in Transactions. Instead, the Division reviewer may rate the manuscript in accordance with his opinion as to its value as Transactions material at the time he submits his recommendation for publication as a proceedings-separate. Review toward Transactions publication will be handled entirely by the Society Committee on Publications.

A total of 30 papers, including 16 carried over from last year, were received by the Committee for publication as Proceedings-Separates. Eighteen were recommended for publication, four were declined, and five were returned to authors for revision. The remaining three were withdrawn, dropped for lack of author cooperation, or referred to another division. In addition, three papers were in the process of review at the time of the ASCE October Convention.

Committee on Design, Construction and Operation of Navigation and Flood Control Locks and Dams

The membership of the committee is as follows:

Charles F. MacNish (Chairman), North Central Division, Corps of Engineers, Chicago

Almern F. Griffin, Special Engineering Consultant, North Central Division, Corps of Engineers, Chicago

Lester W. Angell, St. Lawrence Seaway Development Corp., Buffalo

William D. Robinson, Dravo Corporation, Pittsburgh

Lawrence H. Burpee, Deputy Chief Engineer, St. Lawrence Seaway Authority, Montreal

William E. Kindel (Secretary), North Central Division, Corps of Engineers, Chicago

The membership of this committee is almost entirely new. During the year a paper jointly prepared by Messrs. Bloor and Oliver (former Chairman and Member of Committee) was presented by Mr. Bloor at the St. Louis meeting.

A meeting of the Committee was held on December 6, 1955, in the offices of the Deputy Administrator of the St. Lawrence Seaway Development Corporation, Buffalo, New York. At that meeting, the Committee made plans to prepare papers on such varied subjects as "Navigation and Flood Control Structures in Ohio River Basin," "Great Lakes Harbors," "St. Lawrence Seaway," "New England Flood Control," "Canadian Harbors" and many others.

Committee on Ports and Harbors

The membership of the Committee is as follows:

Frank W. Herring (Chairman), Deputy Director for Comprehensive Planning, The Port of New York Authority, New York

H. W. McCurdy, President, Puget Sound Bridge and Dredging Co., Seattle

C. W. Garrison, Chief Engineer, Allen N. Spooner & Son, Inc., New York

William F. Heavey, Great Lakes St. Lawrence Assn., Washington

R. R. Shoemaker, Chief Harbor Engineer, Port of Long Beach, Long Beach

G. T. Treadwell, Chief Engineer, Port of Seattle, Seattle

The membership of this committee was entirely changed during last year. Chairman Herring is corresponding with his committee for the purpose of exploring the more important topics in the field of ports and harbors requiring the attention of the committee. It is hoped to narrow down the field to one or two important topics to which the committee may devote attention for the immediate period ahead.

Committee on Coastal Engineering

The membership of the committee is as follows:

Richard O. Eaton (Chairman), Chief Technical Advisor, Beach Erosion Board, Corps of Engineers, Washington

Herbert C. Gee, Partner, Gee and Jenson, Consulting Engineers, Palm Beach

Joseph W. Johnson

Harry O. Locher, Sec'y-Treas., The National Association of River and Harbor Contractors, New York

James K. Rankin, Asst. Chief Engineer, State Board of Commerce and Navigation, Newark, New Jersey

Mr. Eaton described the program of research in the subject of coastal engineering, advising that his Committee on Coastal Engineering had discussed the subject with a member of the ASCE Committee on Research. The Committee presented an excellent all-day session at the October Convention. At the Executive Committee meeting, Mr. Eaton suggested joint sponsorship of sessions at the 1956 Annual Meeting in Pittsburgh, with the Committee on Tidal Hydraulics of the Hydraulics Division. Each committee would be responsible for an equal number of papers. Mr. Eaton will discuss it further with Mr. Clarence Wicker, Chairman of the Committee on Tidal Hydraulics.

Committee on Cooperation with Local Sections

The membership of the committee is as follows:

Samuel D. Stickle (Chairman 1954-55), Great Lakes Dredge and Dock Company, New York

Norman R. Moore, Chief, Engineering Division, Mississippi River Commission, Vicksburg

Floyd D. Peterson, Chief, Civic Works, Executive Office of the President, Washington

John G. Turney, Consulting Engineer, Houston

Committee Members Who Are Local Section Representatives:
J. B. Converse (Alabama); Howard A. Preston (Columbia); G. A. Smith (Indiana); Andre L. Jorissen (Ithaca); J. Thornton Starr (Maryland); Herbert C. Gee (Miami); Richard H. Brownley (Mid-South); Charles Smallwood, Jr. (North Carolina); Albert E. Coldwell (Oklahoma); Francis G. Christian (Sacramento); Walter F. Lawlor (St. Louis); Owen G. Stanley (San Francisco); Clifton T. Barker (Tennessee Valley-Knoxville Branch); Henry J. Miles (Texas-Brazos County Branch).

Mr. S. D. Stickle, former Chairman of the Committee, reported at the Executive Committee meeting that in response to a written inquiry, fourteen local sections have submitted the name of a local representative of the Waterways Division who could help to arrange programs and serve as a local contact. With this step taken, Mr. Stickle then expressed the desire to resign as Chairman of the Committee. His resignation was accepted by the Executive Committee with regret and an expression of appreciation for his assistance. A new chairman has not yet been appointed.

Committee on the Regulation and Stabilization of Rivers

The membership of this committee is as follows:

Leo M. Odom (Chairman), Consulting Engineer, Baton Rouge

Lawrence B. Feagin, Chief, Operations Division, Mississippi River Commission, Corps of Engineers, Vicksburg

Joseph F. Friedkin, Principal Supervising Engineer, U. S. Section, International Boundary and Water Commission, U. S. and Mexico, El Paso

Raymond L. Huber, Civil Engineer, Corps of Engineers, Omaha

Charles Senour, Consulting Engineer, Coral Gables, Fla.

Lorenz G. Straub, Director, St. Anthony Falls Hydraulic Laboratory, Minneapolis

Division Newsletter

Austin E. Brant, Jr. (Editor), Tippetts-Abbett-McCarthy-Stratton, New York

A REMINDER

Members of the Society are now permitted to enroll in two technical divisions and receive all the publications of each. This is in line with the recommendations of the Executive Committee of the Waterways Division, which has been active since last March in urging that members be permitted to affiliate with more than one division. Send in the coupon in any of the recent issues of "Civil Engineering" as soon as possible.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

VOLUME 81 (1955)

MARCH: 634(PO), 635(PO), 636(PO), 637(PO), 638(PO), 639(PO), 640(PO), 641(PO)[c], 642(SA), 643(SA), 644(SA), 645(SA), 646(SA), 647(SA), 648(ST), 649(ST), 650(ST), 651(ST), 652(ST), 653(ST), 654(ST)[c], 655(SA), 656(SM)[c], 657(SM)[c], 658(SM)[c].

APRIL: 659(ST), 660(ST), 661(ST)[c], 662(ST), 663(ST), 664(ST)[c], 665(HY)[c], 666(HY), 667(HY), 668(HY), 669(HY), 670(EM), 671(EM), 672(EM), 673(EM), 674(EM), 675(EM), 676(EM), 677(EM), 678(HY).

MAY: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(HY), 684(ST), 685(SA), 686(SA), 687(SA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(PO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

JUNE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(SA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(SM)[c], 760(WW)[c].

AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

OCTOBER: 809(ST), 810(HW)[c], 811(ST), 812(ST)[c], 813(ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(BD), 858(BD), 859(BD), 860(BD).

VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1), 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877(HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(HY1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1), 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(HY1)[c], 904(PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913(WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

c. Discussion of several papers, grouped by Divisions.

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